ABSTRACT

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	Ning Yang, Ph.D., 2010
Directed By:	Professor Paul Schonfeld, Department of Civil and Environmental Engineering

With the increase of the number, duration, and scope of maintenance projects on the national highway system, transportation agencies face great challenges in developing effective comprehensive work zone management plans which minimize the negative impacts on road users and workers. The types of maintenance operation, timing, duration, configuration, and user impact mitigation strategies are major considerations in developing work zone management plans. Some of those decisions may not only affect road users during the maintenance phase but also have significant impacts on pavement serviceability in future years.

This dissertation proposes a systematic methodology for jointly optimizing critical work zone decisions, based on analytical and simulation models developed to estimate short-term impacts during the maintenance periods and long-term impacts over the pavement life cycle.

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14. ABSTRACT

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OPTIMIZATION OF HIGHWAY WORK ZONE DECISIONS CONSIDERING SHORT-TERM AND LONG-TERM IMPACTS

By

Ning Yang

Dissertation submitted to the Faculty of the Graduate School of the
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Advisory Committee:
Professor Paul Schonfeld (ENCE, UMCP), Chair
Professor Gang-Len Chang (ENCE, UMCP)
Associate Professor Dimitrios G. Goulias (ENCE, UMCP)
Professor Thomas M. Corsi (BMGT, UMCP)
Professor Steven I-Jy Chien (ENCE, NJIT)

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Table of Contents

List of Tables

List of Figures

List of Symbols

a	-	A constant parameter which controls the degrees of curvature of the pavement performance curve
a_i	-	The layer coefficient of the i^{th} layer
a_a	mph/s	Average acceleration rate
b	-	A constant parameter which controls the threshold pavement condition value in the pavement performance curve
$AADT$	veh/day	Annual average daily traffic
$AADT_m$		Designed AADT for baseline year when rehabilitation is conducted
c_0	vph	Maximum discharge rate without work zone
c_w	vph	Maximum discharge rate with work zone
C_T	$/project	Total work zone cost of the project
C'_T	$/period	Total work zone cost in one cyclic period
$C_{A,i}$	$	Agency Cost of the i^{th} work zone
$C_{U,i}$	$	User Cost of the i^{th} work zone
$C_{M,i}$	$	Agency Maintenance Cost of the i^{th} work zone
$C_{S,i}$	$	Agency Traffic Mitigation Cost of the i^{th} work zone
$C_{I,i}$	$	Agency Equipment/Labor Idling Cost of the i^{th} work zone
$C_{D,i}$	$	User Delay Cost of the i^{th} work zone
$C_{V,i}$	$	User Vehicle Operating Cost of the i^{th} work zone
$C_{E,i}$	$	User Expected Accident Cost of the i^{th} work zone
C_{LT}	$	Long-term future cost
C_{LM}	$/year	Annual maintenance cost
C_{CL}	$/year-lane-mile	EUAC per lane-mile
C_d	-	Drainage coefficient
CEI	-	Cost-effectiveness index
CI	-	Cost index
CL	year	Pavement life cycle length
D_T	hr/project	The maximum duration of a cyclic period
D'_T	hr/period	The duration of a cyclic analysis period
D_i	hr	The duration of the i^{th} work zone
D_D	veh.hr	The delay caused by a work zone
D^m	veh.hr	The delay of the traffic on the mainline
D^d	veh.hr	The delay of the original traffic on the detour
D^p	veh.hr	The delay of the traffic diverted from mainline to the detour
D_d^m	veh.hr	The deceleration delay on the mainline
D_m^m	veh.hr	The moving delay on the mainline
D_a^m	veh.hr	The acceleration delay on the mainline
D_q^m	veh.hr	The queuing delay on the mainline
D_r^m	veh.hr	The systematic delay on the mainline
D	inch	Thickness of pavement slab
D_L		lane distribution factor
E_i	-	Ending time of the i^{th} work zone
E_1	-	ESAL factor for passenger cars
E_2	-	ESAL factor for Single Unit Trucks
E_c	psi	Modulus of elasticity of PCC
EI	-	Effectiveness Index

EV_i		ESAL factor of the i^{th} vehicle classs
EUAC	$/year	Equivalent Uniform Annual Costs
f_2	-	The multi-lane operation cost saving factor
f_4	-	The multi-lane operation time saving factor
F_{ESAL}	-	ESAL factor
GR	%	Traffic Growth Rate
H	mile	Headway between lead and follower vehicles
H_j	mile	Jam density headway
i	%	Interest Rate
J	-	Load transfer coefficient
k_m	lb/in.^{3}	Modulus of subgrade reaction
k	hr	Driver sensitivity factor for the follower vehicle
l_{veh}	ft/veh	Average vehicle length
L_T	lane-mile	Total lane-mile to be maintained in the project
L'_T	lane-mile/period	Total lane-mile maintained in one cyclic period
L_{wi}	mile	Length of the i^{th} work zone
L_m	mile	Travel distance along mainline route
L_d	mile	Travel distance along detour route
L_{AB}	mile	Length of Segment AB
L_{AC}	mile	Length of Segment AC
L_{CD}	mile	Length of Segment CD
L_{DB}	mile	Length of Segment DB
m	#	Total number of work zones needed to complete the project
m'	#/period	Total number of work zones set up in one cyclic period
m_i	-	The layer drainage coefficient of the i^{th} layer
M_R	psi	Subgrade resilient modulus (psi)
n	year	Number of years of the analysis period
N	#/direction	Number of lanes in one direction
N_{wi}	#	Number of closed lanes in the i^{th} work zone
N_{ai}	#	Number of access lanes in the i^{th} work zone
N_{ci}	#	Number of crossover lanes in the i^{th} work zone
N_L	#	The number of new layers
N_{AB}	#/direction	Number of lanes in Segment AB (direction with work zone)
N_{AC}	#/direction	Number of lanes in Segment AC (direction with work zone)
N_{CD}	#/direction	Number of lanes in Segment CD (direction with work zone)
N_{DB}	#/direction	Number of lanes in Segment DB (direction with work zone)
NV	#	the number of vehicle classes
NPV	$	Net Present Value
$p'(t)$	%	Adjusted traffic diversion rate at time t
$p(t)$	%	Natural diversion rate at time t
P_m	-	Design serviceability index
P_t	-	Threshold serviceability index
$P(t)$	-	Serviceability index at time t
PV_i		percentage of the i^{th} vehicle class
$Q(t)$	vph	Time-varying traffic flow volume
$Q_{p,max}$	vph	Maximum allowed diverted volume
$q(t)$	veh	Time-varying number of vehicles in queue at time t
q_m	veh	Maximum acceptable number of vehicles in queue
$q_{L,max}$	mile	Maximum acceptable queue length
S_i	-	Starting time of the i^{th} work zone

s_d	mile	Average deceleration distance
S_F	mile	Distance traveled by follower vehicle to come to a complete stop
S_L	mile	Distance traveled by lead vehicle to come to a complete stop
S_0	-	Combined standard error of the traffic prediction and performance prediction
S'_c	psi	Modulus of rupture of PCC
SN	-	Structure number
T_s	-	Starting time of a cyclic period
T_e	-	Ending time of a cyclic period
T_{int}	hr	Average waiting time passing intersections along the detour
T_i	inch	The thickness of the i^{th} layer
V	mph	Speed of the follower vehicle in a car-following model
ΔV	mph	The difference in speed between lead and follower vehicles
V_f	mph	free-flow speed
V_w	mph	Average work zone speed
V_{AB}	mph	Free flow speed in Segment AB
V_{AC}	mph	Free flow speed in Segment AC
V_{CD}	mph	Free flow speed in Segment CD
V_{DB}	mph	Free flow speed in Segment DB
W_{18}	-	Predicted number of 18 kip Equivalent Single Axle Load (ESAL)
$\vec{X}_S$	-	Short-term work zone decisions
$\vec{X}_L$	-	Long-term work zone decisions
z_1	$/zone	The fixed setup cost per work zone
z_3	hr/zone	The fixed setup time per work zone
z_2	$/lane.mile	The unit length maintenance cost
z_4	hr/lane.mile	The unit length maintenance time
Z_R	-	Standard normal deviate
β_1	$	The fixed employment cost of a traffic management strategy
β_2	$/hr	The unit-time employment cost of a traffic management strategy
v_I	$/hr	The average cost of idling crews and equipments
v_D	$/hr	The average value of time per vehicle
v_s	$/cycle	The average VOC per speed change cycle
v_d	$/mile	The average VOC per unit distance
v_q	$/veh.hr	The unit queue idling VOC per vehicle
v_E	$/accident	The average cost per crash
γ_E	acc/100mvh	The estimated number of crashes per 100 million vehicle hours of travel
δ_w	%	Adjustment of the work zone capacity
δ_d	%	Adjustment of the detour capacity
δ_p	%	Adjustment of traffic diversion percentage
ΔR		Distance traveled by follower vehicle during its reaction time
ΔS		traveled by follower vehicle over time interval Δt
ΔPSI		The difference between the restored design serviceability index and threshold serviceability index, $\Delta PSI=Pm-Pt$

Chapter 1 Introduction

Chapter 1 provides a concise profile of this dissertation, describing the motivation for this research, stating the research problem, and describing the technical approaches proposed to accomplish the research goal. The organization of the dissertation is presented at the end of this chapter.

1.1 Research Motivation

The prosperity and economic growth experienced in the United States during the 1990s contributed to an increase in demand for many modes of surface travel. However, a significant fraction of the nation’s current highway system has been in poor, mediocre, or fair condition (*TRB NCHRP 330, 2004*). The deterioration of the national highway system severely affected wear-and-tear on vehicles, fuel consumption, travel time, congestion, comfort and public safety. To sustain highways in a safe and usable condition, state and federal transportation agencies have increased the number, duration, and scope of maintenance activities in recent years.

Since conducting maintenance work in work zones usually reduces the available roadway capacity and thus forms traffic bottlenecks, state and federal transportation agencies are facing great challenges in completing maintenance projects efficiently and economically while also maintaining work zone safety and mobility and minimizing the traffic disruption. To accommodate these needs, there is currently a rising trend of encouraging construction engineers, traffic engineers, safety experts and other technical specialists to work together on developing a comprehensive work zone management plan in the early planning stage (*FHWA, 2005*). The timing, duration of

work, lane closure configuration, traffic management strategies and type of construction operation are major considerations in developing a successful work zone management plan. Most of these decisions only affect agency and road user cost over construction phase and therefore are considered as short-term work zone decisions. There are also so-called long-term decisions, such as construction operation type, whose impact on pavement serviceability in the future years should not be neglected. Consequently life-cycle cost analysis is required in the decision making process if long-term decisions are involved.

While traffic varies by day of week and time of day and numerous candidate transportation management strategies exist, it is a challenging task to design the most appropriate work zone management plan and poor decisions can be quite costly. Although the comparison of several competing alternatives is widely used to solve this problem in practice, it becomes an inefficient approach when the scale and complexity of the project increase. To ease designer’s work load as well as improve the decision making process, extensive researches have been conducted on the development of optimization tools which can automatically explore high-quality work zone plans. In previous studies, the selection or optimization of short-term decisions and that of long-term ones are relatively separate. The former are mainly explored in the traditional traffic management area and the latter constitute a popular topic in the pavement management field. Although the above decisions can be highly interrelated, few comprehensive methodologies have been developed to bind the optimizations of those two kinds of work zone decisions in one analytic framework. It is worthwhile to develop appropriate work zone analysis tools which can aid highway agencies in

developing cost-effective highway maintenance or rehabilitation plans, especially when both types of decisions are made jointly in one comprehensive management plan.

1.2 Problem Statement

The general work zone decision optimization problem studied in this dissertation can be defined as the follows:

A road section with the length L and the number of lanes N is required to be maintained within the time period [T_s, T_e]. Given necessary project data, geometric data, traffic data, and a set of candidate work zone impact mitigation strategies, decision makers seek to design the most cost-effective work zone management plan which can provide appropriate answers to the following questions:

(1) Which maintenance intensity level should be chosen (e.g. overlay thickness) in order to obtain an economical durability level?

(2) What is the optimal resurfacing frequency and resurfacing time intervals?

(3) How should the road section be divided into work zones? How long and wide should each work zone be?

(4) At what times should the lanes in each work zone be closed and reopened to traffic under time-varying traffic inflows?

(5) What traffic impact mitigation strategies, such as accelerated work, detours and temporary traffic control measures, should be implemented in the maintenance project?

When all decisions taken into account only have temporary impacts during the maintenance phase, the best work zone management plan is defined as the one that

minimizes the one-time total cost (C) consisting of the monetary expense that transportation agencies spent on performing the maintenance work (C_M) and the added cost to roadway users resulting from maintenance activity (C_U).

When pavement strategies which may yield dissimilar pavement performance levels in the future years are considered, a work zone management plan has to be evaluated from a long-term point of view. The optimal plan is the one which achieves the highest Cost-Effectiveness Index (*CEI*). The value of CEI is calculated based on the benefits received by users and the cost to provide those benefits over the life cycle of the maintained pavement.

1.3 Research Objectives

With a great variety of candidate work zone management design options, highway agencies face the tasks of choosing the most appropriate combination which can obtain the desired results effectively and economically. Although the comparison of several competing alternatives based on subjective engineering judgment is widely used to solve this problem, it becomes an inefficient approach when the scale and complexity of the project increase. To ease highway agencies' work load as well as improve the above decision making process in early planning stages, this dissertation aims to develop a systematic decision aid tool which can integrate short-term and long-term work zone decisions into one comprehensive optimization framework. The proposed methodology focuses on maintenance projects for multiple-lane highways under time-dependent inflows in a network with possible detours. This research should

provide a comprehensive decision support tool capable of evaluating different work zone management plans as well as automatically searching for high quality solutions.

The decision support tool will include two maintenance plan evaluation models:

(1) The model for evaluating short-term impacts of work zone decisions, measured by one-time total cost (C_T);
(2) The model for evaluating long-term impacts of work zone decisions, measured by the Cost-Effectiveness Index (CEI).

Consequently, two optimization models will be formulated based on the type of work zone plan decisions to be optimized:

(1) The mathematical model to optimize short-term decisions;
(2) The mathematical model to jointly optimize short-term and long-term decisions.

A computationally efficient solution search method will be developed to find satisfactory solutions to the optimization problem.

1.4 Research Methodology

This research starts from identifying critical decisions that transportation agencies need to make during the development of a work zone management plan. Cost assessment models are then formulated as functions of relevant decision variables for calculating agency costs (C_A) and user costs (C_U) associated with performing highway maintenance.

The agency cost is the expense of suppliers on the maintenance of a highway segment or network within the study scope during the analysis period. The agency costs considered in this study include maintenance cost (C_M), traffic mitigation cost (C_S), and equipment/labor idling cost (C_I) required to perform maintenance as well as annual routine maintenance cost (C_{LM}) over the performance life of the newly maintained pavement. The user cost consists of user delay cost (C_D), added vehicle operating cost (C_V) and expected accident cost (C_E) resulting from traffic disruption caused by maintenance activity. Long-term user vehicle operating costs under normal conditions are not directly considered in the study due to the difficulty of establishing and calibrating models relating this cost component with pavement conditions. User delay cost plays a key role in user costs. Two approaches are employed to estimate the user delay costs. One is a simulation approach, which uses CORSIM, a commercial simulation program, to simulate work zone conditions; the other is an analytical method, several parts of which are derived from simulation results.

Since different measures of effectiveness (MOEs) are used to evaluate a work zone management plan depending on whether or not long-term pavement decisions (e.g., type of maintenance and overly thickness) are involved, two kinds of optimization models are developed. If only considering short-term impacts during maintenance phase, the work zone management optimization problem is formulated as a minimization problem of finding feasible solutions that obtain the least one-time work zone total cost (C_T), which is the sum of work zone agency costs (C_A) and user costs (C_U). When long-term impacts over the pavement life cycle are used as MOE, a maximization problem is formulated with the objective of maximizing the

Cost-Effectiveness Index, which is the ratio of Effectiveness Index (EI) quantifying user benefits and Cost Index (CI) quantifying life cycle cost. The estimation of Effectiveness Index (EI) is based on a deterministic pavement performance model. The optimization problems are subject to a series of work zone operation and traffic impact constraints (e.g. project deadline, maximum acceptable queue length). The objective functions are expressed in Eq. 1-1 and Eq. 1-2, where short-term and long-term decision variables are denoted by $\vec{X}_S$ and $\vec{X}_L$, respectively.

Short-term Work Zone Optimization Model

Min $\quad C_T(\vec{X}_S) = C_A(\vec{X}_S) + C_U(\vec{X}_S)$ Eq. 1-1

s.t. (1) work zone operation constraints
(2) traffic impact constraints

Long-term Work Zone Optimization Model

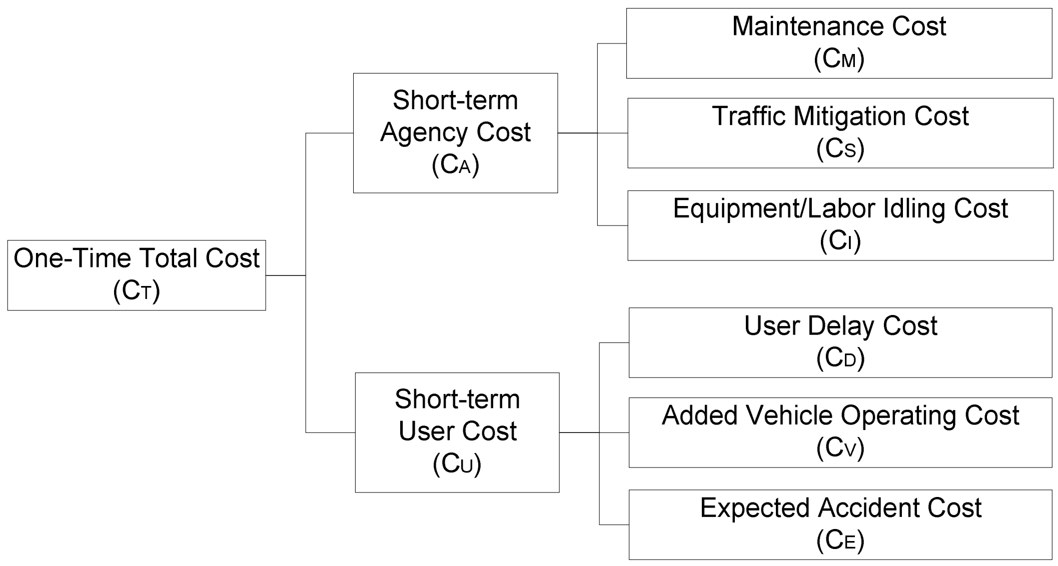
Eq. 1-2

Max $\quad CEI(\vec{X}_S, \vec{X}_L) = \dfrac{EI(\vec{X}_L)}{CI(\vec{X}_S, \vec{X}_L)}$

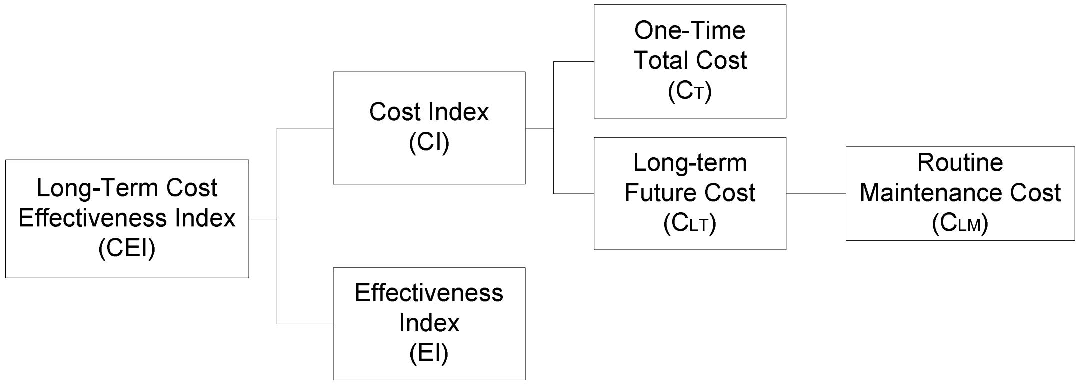
s.t. (1) work zone operation constraints
(2) traffic impact constraints

where, EI = Effectiveness Index quantifying user benefits;
CI = Cost Index quantifying Life cycle cost.

All the cost components of the C_T and CEI considered in this study are listed in Figure 1-1. The analysis framework is illustrated in Figure 1-2.



(a) Components of One-Time Total Cost (C_T)



(b) Components of Long-Term Cost Effectiveness Index (CEI)
Figure 1-1 Cost Components of C_T and CEI

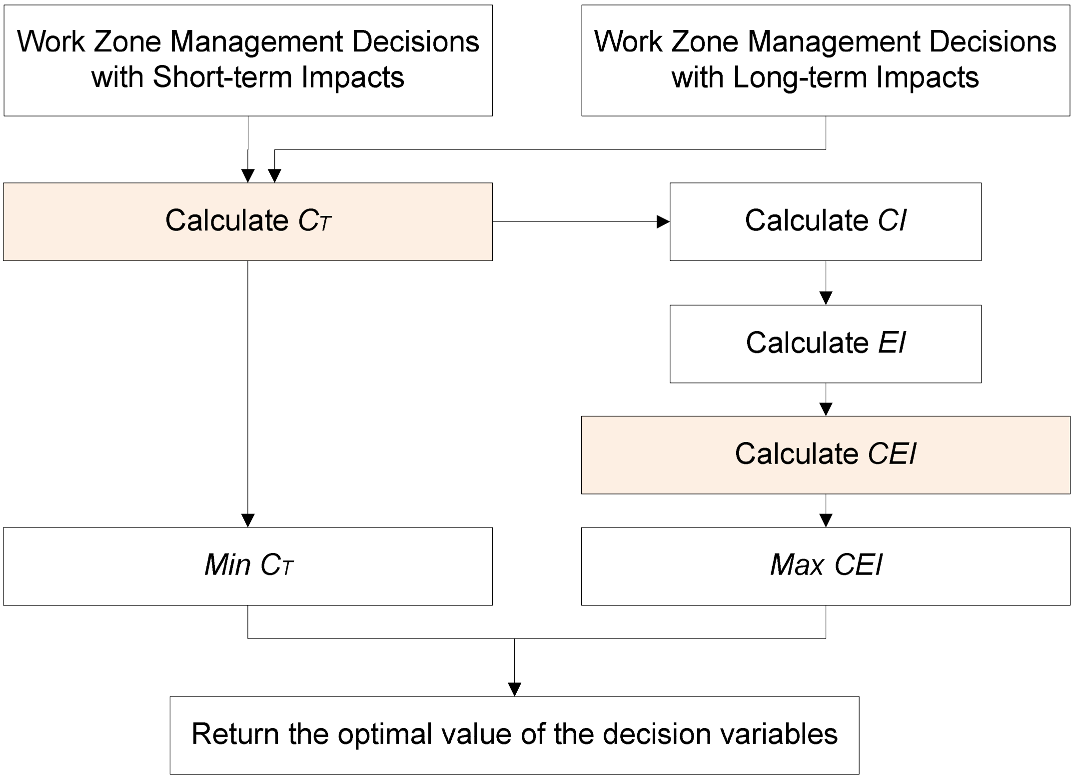


Figure 1-2 Analysis framework

When the user cost is obtained from the analytic method for time-dependent traffic inflows, the complex and combinatorial nature of the mathematical formulation of the objective functions precludes conventional analytical solution algorithms. When the simulation method is used, there is no closed-form expression for the objective functions. Therefore, a heuristic optimization algorithm will be developed to search the solution space and find an optimized solution to the work zone decision optimization problem.

Since optimization based on simulation is very time-consuming, two approaches will be applied to speed up the optimizing search process when the simulation is used to evaluate the objective function. One is a hybrid algorithm that will pre-optimize the decision variables analytically before performing the optimization based on detailed

simulation. The other is parallel computing which distributes objective function evaluation tasks to multiple processors.

1.5 Organization of Dissertation

To achieve the above research purposes, this dissertation will consist of the following seven chapters. The interrelations among these chapters and their development sequence are shown in Figure 1-3. The focus of each chapter is detailed below.

Chapter 1, “Introduction,” presents background information, research objectives, and the technical approach used in this research.

Chapter 2, “Literature Review,” focuses on reviewing research performed in the previous twenty years that is considered to be relevant to the project objectives. Based on the review results, the potential contributions of this research work are addressed.

Chapter 3, “One-Time Work Zone Cost Model,” describes critical short-term work zone decisions faced by the decision makers. Analytical models are developed to estimate the one-time total cost associated with performing maintenance work on a road section in a simplified network.

Chapter 4, “Optimization of Short-term Decisions based on Analytic Model,” formulates the work zone decision optimization model considering short-term impacts. The one-time work zone cost, as the minimization objective, is evaluated with the analytical model developed in Chapter 3. A heuristic optimization algorithm called two-stage modified simulated annealing (2PBSA) is developed to solve the work zone optimization problem.

In Chapter 5, “Optimization of Short-term Decisions based on Simulation,” short-term decisions are optimized with the one-time total cost evaluated by simulation. A hybrid methodology, in which both the analytic model and the simulation model are used to evaluate the total cost, is developed to speed up the optimization process. A parallel computing technique is also applied to further reduce the computational time of simulation-based optimization.

Chapter 6, “Joint Optimization of Short-term and Long-term Decisions,” presents the formulation of a Cost-Effectiveness Index used to measure the performance of work zone management plans with long-term impacts. An optimization framework is developed to jointly optimize short-term and long-term work zone decisions with the objective of maximizing the Cost-Effectiveness Index.

The conclusions, research contribution and recommendations for future studies will be summarized in Chapter 7.

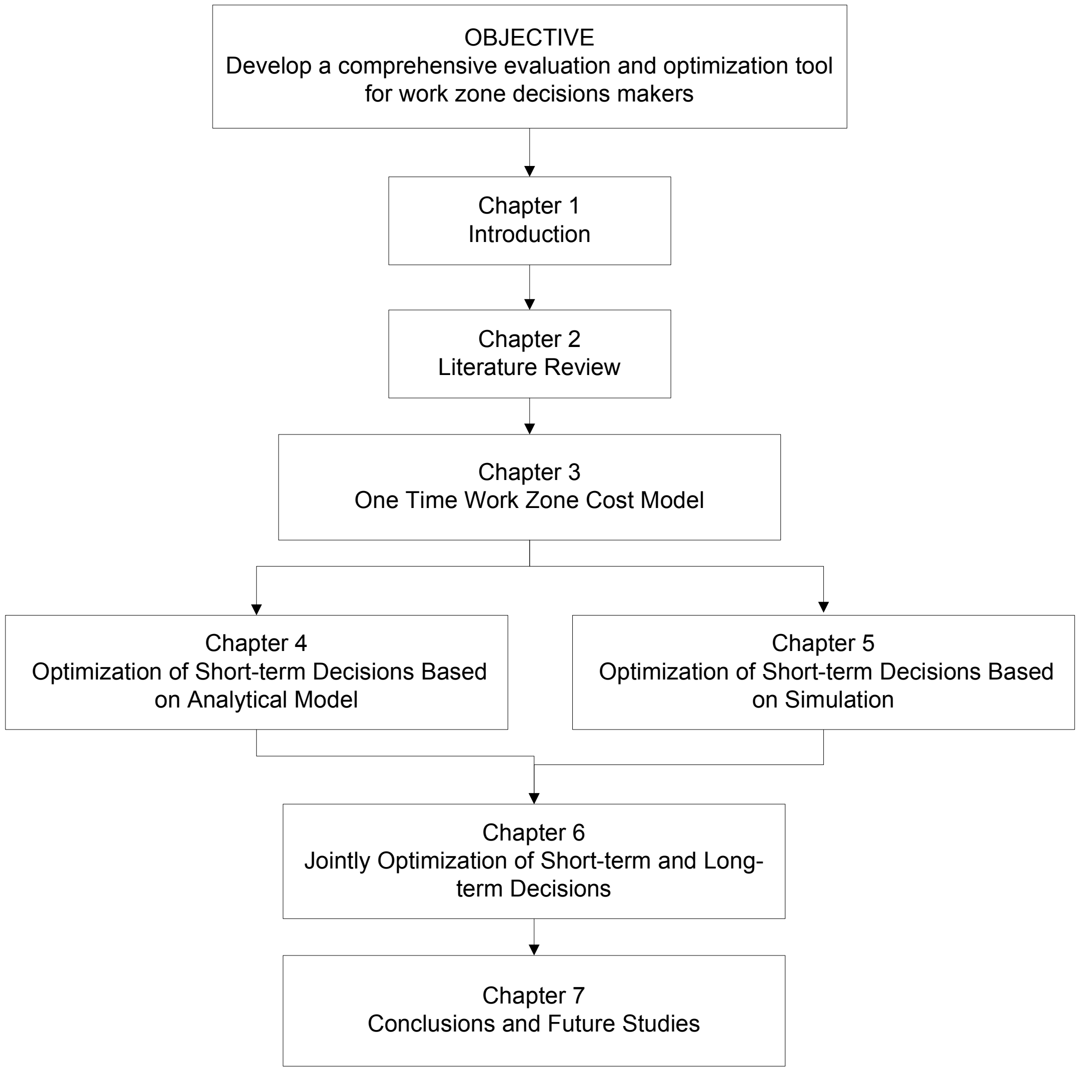


Figure 1-3 Organization of the Dissertation

Chapter 2 Literature Review

The purpose of this chapter is to concisely review the relevant literature on the operation, management and optimization of highway maintenance. This chapter is organized into five main parts: (1) maintenance operation and management, (2) maintenance cost estimation, (3) work zone delay estimation, (4) long-term and short-term decision optimization, and (5) simulation-based optimization.

The first step in developing an optimization model is to identify decision variables. Thus it is important to know the critical issues in road managing agencies' decision making process during the plan and implement of highway maintenance activities. Those issues will be discussed in the first section of this chapter.

The concept of "optimal" decisions closely linked to the formats of cost-effectiveness indicators. The most common are Net Present Value (NPV) (*Tsunokawa et al., 2006*), the total cost (*Mamlouk et al., 2000; Elbehairy et al., 2006; Chen and Schonfeld; 2007*), the Benefit Cost Ratio (B/C) (*TRB NCHRP 523, 2004; Pasupathy, 2007*) or some specific index (*Mohammadi et al., 1995; Bosurgi and Trifiro, 2005*) In this dissertation, the long-term or short-term total cost, with additive function, is considered the appropriate economic efficiency indicator of choice because this research aims at evaluating project-level alternatives that result in equal benefits but entail unequal costs. Therefore, state-of-art of the maintenance cost assessment methods are presented in the second section. As an important cost component in this study, the work on estimating work zone user delay costs are reviewed in the third section.

It is natural for researchers to relate the design of maintenance plans or the selection of maintenance alternatives with the cost-effectiveness evaluation through optimization methods. Previously completed work on this topic are reviewed and assessed in the fourth section.

Since the proposed optimization model is designed to use simulation as a tool for analyzing traffic impact, the literature search also includes topics relevant to the optimization based on simulation. The fifth section presents the findings relevant to such optimization.

2.1 Operation and Management of Maintenance Work

2.1.1 Definition of Maintenance

There is a wide divergence amongst the professional communities and the general public on the precise scope and definition of "maintenance". In practice, the dividing line between maintenance and rehabilitation is particularly blurred. For some professionals, maintenance means only relatively low-cost treatments helping slow the rate of deterioration by identifying and addressing specific pavement deficiencies (e.g., seal coats, cracking sealing, patching, joint sealing, grinding, milling, and grooving); more aggressive action of repairing portions of an existing pavement to reset the deterioration process (e.g., overlay, removing and replacing the wearing course) is termed "rehabilitation" or "reconstruction" (*Muench, 2003; TRB NCHRP 330, 2004; Simpson and Thompson, 2006*).

In this study, maintenance is defined as "set of activities required to keep a component, system, infrastructure asset, or facility functioning as it was originally designed and constructed to function" (*Hudson et al., 1997*). Treatment approaches with different work intensities, including preservation, rehabilitation, restoration and reconstruction, can all fall under this general definition of maintenance. Maintenance may be preventive or corrective, as well as routine or major, planned or reactive (*TAC, 1997*)

2.1.2 Highway Maintenance Management

Making effective decisions about highway maintenance requires appropriate management, which means ensuring that proper maintenance treatment is applied at the proper time in the correct place over a planning horizon (*O'Flaherty, 2002*). The decision-making process must take a long-term view of the economic life of a highway section, reflecting the transportation agency's long-term responsibility.

The management of highway maintenance is a complex process, which has been described and developed at two levels: network and project levels. Network-level analysis deals with the network as a whole and is generally concerned with high-level decisions relating to network-wide planning, policy, project prioritization, and resource allocation. Project-level analysis mainly involves the evaluation of competing alternatives for constituent sections within the roadway network. It aims at finding the optimal strategy that achieves the maximum economy (*Ozbay, 2003*). Since the research undertaken in this dissertation targets the optimization of maintenance decision at project-level, the literature search focuses on issues in the project-level analysis.

An important project-level maintenance decision is to choose appropriate work intensities, materials, and techniques, for preserving or improve the roadway's service level in a cost-effective way. Work intensity options can be minor maintenance, major maintenance, minor rehabilitation, major rehabilitation and reconstruction, which will result in different reset pavement condition and durability (Figure 2-1). Material, technique and other detailed options can also be considered. For instance, the overlay thickness should be determined when Asphalt Concrete (AC) overlay is selected as the rehabilitation method for a roadway section.

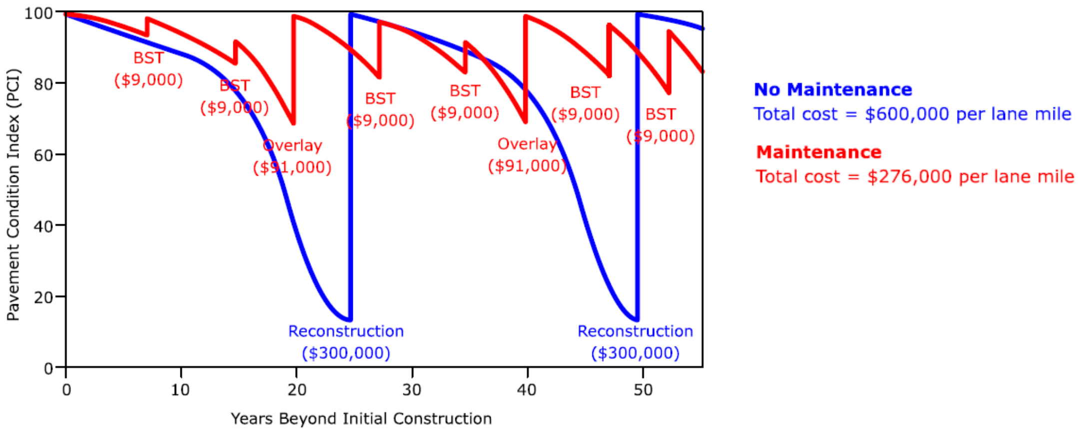


Figure 2-1 Effects of Maintenance Activities on the Pavement Condition (WSDOT, 1995)

Coupled with selecting maintenance options, determining the timing to program maintenance actives is another critical long-term decision. The best way to preserve the road condition is by restoring the pavement to its original condition every year. However, its cost is substantial and not affordable for agencies considering traffic disturbances and budget limitations. Another extreme is to use the road to the fullest extent without performing any periodic maintenance, leading to frequent reconstructions within the analysis period. This option also results in unacceptable

overall cost as reconstruction is quite expensive. Therefore, the cost-effective scheduling of pavement maintenance is required.

2.1.3 Work Zone Management

A work zone is defined in the Highway Capacity Manual as an area of a highway in which maintenance and construction operations are taking place that impinges on the number of lanes available to traffic and affect the operational characteristics of traffic flowing through the area (*TRB, 2000*). The short-term impact caused by maintenance work depends on the characteristics of the work zones and the highway traffic conditions. Work zone characteristics of concern include such factors as work zone length, number and capacity of lanes open, duration of lane closures, timing of lane closures, posted speed, and the availability of alternative routes.

Work zone length is an important issue that has been relatively neglected. In general, longer zones tend to increase the user delays, but the maintenance activities can be performed more efficiently (i.e., with fewer repeated setups) in longer zones (*Schonfeld and Chien, 1999*). In practice, such lengths have been usually designed to reduce costs to highway agencies rather than users.

The lane closure type is one of the major factors which affect the vehicle capacity in work zones and it also affects agency costs to a considerable degree. There are three main lane closure types for work zones: partial lane closure, full lane closure and crossover (*Pal and Sinha, 1996*). In a partial lane closure one or more lanes are closed in one or both directions, but not all the lanes in one direction are closed simultaneously. Traffic cones, drums, or concrete barriers are used to close the lanes, and maintenance

and rehabilitation activities are performed on the closed lanes. During full road closure, traffic is detoured, allowing full access to roadway facilities. Under the appropriate conditions, a full closure can be an effective way to complete projects with shorter duration and less safety risks. Departments of Transportation in Oregon, Kentucky, Michigan, Ohio, Washington, and Delaware have experience in using a full closure approach to conduct road rehabilitation/reconstruction projects (*FHWA-OP-04-009, 2003*). In a crossover arrangement, the traffic that normally uses the roadway is crossed over the median and two-way traffic is maintained on the other side of the roadway (*Jiang, 1999*). Successful crossover operation can fully utilize the remaining capacity in the opposite direction. In addition, in a crossover lane closure strategy sufficient working spaces are available, which may improve the safety of the workers and increase their productivity as well as the quality of their work. However, due to the additional cost of constructing the crossover facility (e.g. concrete barriers), the fixed set up cost in cases of crossover is always higher than in cases of partial lane closure at sites. It is noted that sometimes closed lanes may include not only maintained lanes, but also additional lanes which are used to provide access to and from the work site for maintenance and construction vehicles or provide buffer space to separate traffic and work sites from safety consideration.

Since travel demands are time-varying, work zone scheduling can greatly affect the traffic impact caused by lane closures. Work zones can be categorized into three designations: (1) short-term sites, at which maintenance work lasts less than one day (24 hours) (*Jiang, 2003*); (2) intermediate sites, at which work lasts over one day but less than four days; (3) long-term sites, at which work lasts more than four days

(*Rouphail et. al., 1988*). Unlike in long-term projects which continuously occupy the road space for several days or months, short term and intermediate work zones are often limited to the time defined in some construction windows, e.g. off-peak daytime, nighttime periods, or weekend periods, in order to avoid the higher volume daytime hours and associated traffic delays.

With the rapidly increasing traffic demand, many highway agencies are under increased pressure to accelerate project completion for mitigating the public's dissatisfaction of the construction speed and traffic congestion relevant to work zones. As a result, nontraditional innovations and technologies speeding up the maintenance operations are gaining transportation engineers' and researchers' attention. Addition to economic measures (e.g. lane rental, contract incentive/disincentive and cost (A) and time (B) bidding), work zone management measures, for instance, use of longer lane closure durations, are also among those technologies. First, the expansion of typical metropolitan peak-period congestion levels from 2-3 hours daily in the early 1980s to 5-6 hours daily today is resulting in a smaller window of opportunity for contractors to perform roadwork without impacting peak period traffic flow (*FHWA, 2003)*. Second, the frequent mobilization and shutdowns required of the construction process inhibit the productivity of short-term closures (*Nam et al., 1999*). Third, due to less careful construction, less curing time for materials and additional transverse joints, short-term lane closures may lead to poor construction quality which, in turn, may affect pavement life expectancy and pavement service level (*Sprinkel, 1993; Lee and Thomas, 2007*).

2.2 Maintenance Cost Estimation

All new construction, reconstruction, rehabilitation and maintenance projects should employ some level of economic evaluation to determine the most cost effective method and timing. Life cycle cost analysis (LCCA) is an economic analysis tool that compares costs attributable to maintenance actions over a specified period of time in selecting optimal projects or implementation alternatives. One or more these costs are usually summed over time by discounting all costs that occur at different times using the present worth method to account for the time value of money. Either a total present worth or an annualized cost can be considered as the cost effectiveness indicator.

2.2.1 Life Cycle Cost

The costs can be classified into two categories: (1) agency costs; (2) user costs, and (3) societal cost.

Agency costs include all costs incurred directly by the agency over the life of the project or a specific planning period. Taking a long-term view, agency costs typically include initial construction cost, future maintenance cost, and the associated administrative cost. In work zone analysis, agency costs are those expenses required to finish the maintenance activities based on the work types. Those normally include labor costs, equipment costs, material costs, traffic control costs, and administration costs. Underwood (*1994*) analyzed the work duration and the maintenance cost per 10,000 m^2 for five different roadway maintenance activities (i.e., surface dressing, asphalt surface, porous asphalt, 10% patching, and milling out). The average

maintenance costs were calculated based on prices quoted to highway authorities in the summer of 1993.

User costs are the delay, vehicle operating, and accident (crash) costs incurred by the users of a facility during the construction, maintenance and everyday use of a roadway section (*Najafi and Soares, 2001*). There are user costs associated with normal operations and work zone operations. The normal operation category reflects highway user costs associated with using a facility during periods without work zone activities. User costs in this category are usually functions of the pavement performance. User costs in work zone operations category reflects highway user costs associated with using a facility during periods with work zones that restrict the capacity of the facility and disrupt normal traffic flow (*Walls III and Smith, 1998*).

2.2.2 Work Zone User Costs

Work zone user costs receive great attention in work zone analysis because they tend to dominate other costs and because community concerns and reactions to work zone activities affect many aspects of work zone decisions.

Work zone user delay costs result from increases in travel time through the work zone from speed reduction, congestion delays, or increased distances as a result of taking a detour (*FHWA, 1989*). Typically, the delay cost can be determined by multiplying the user delay by the value of user time (*Walls III and Smith, 1998*). Studies on user delay estimation will be introduced in the next subsection.

Vehicle operating costs are the costs associated with owning, operating, and maintaining a vehicle including: fuel consumption, tire wear, maintenance and repair,

and so on. Many factors such as vehicle characteristics, vehicle speed, road geometrics, road surface type and condition, environmental factors can affect vehicle operating costs. Vehicle operating costs can be formulated empirically or mechanistically, deterministically or probabilistically. In many studies, vehicle operating costs were formulated using classical regression analysis of historic information or simulation results (*Booz Allen & Hamilton, 1999; Berthelot et. al.., 1996; Vadakpat et.al., 2000*).

Accident (crash) costs are related to the historical crash rate, vehicle miles of travel, delay, work zone configuration, and average cost per crash. Crash rates are commonly specified as crashes per 100 million vehicle miles of travel (100 M VMT). Overall crash rates for the various functional classes of roadway are fairly well established. Crash rates for work zones, however, are not easy to estimate due to the limited amount of data and the variety of work zone types. McCoy et al. (*1980*) found the average rate was 30.8 crashes per 100 million vehicle miles (acc/100 mvm) on I-80 in Nebraska between 1978 and 1984. Pigman and Agent (*1990*) found that the work zone crash rate varied from 36 to 1,603acc/100 mvm on different highways based on the crash data collected from the Kentucky Accident Reporting System (KARS) for the 1983-1986 periods. Chien and Schonfeld (*2001*) determined work zone crash cost from the product of the number of crashes per 100 million vehicle hours multiplied by the the increasing delay, and the average cost per crash.

2.3 Work Zone Delay Estimation

The delays related to work zones can be classified into five categories (*Jiang, 1999; NJDOT, 2001*): (1) deceleration delay by vehicle deceleration before entering a work

zone, (2) moving delay by vehicles passing through work zones with lower speed, (3) acceleration delay by vehicle acceleration after exiting work zone, (4) queuing delay caused by the ratio of vehicle arrival and discharge rates, and (5) Detour Delay by the additional time necessary to traveling the excess distance the detour imposes.

Over the years a number of manual and computerized approaches have been developed for estimating the work zone delays (*McCoy and Peterson, 1987; Schonfeld and Chien, 1999; Venugopal and Tarko, 2000; Chien and Schonfeld, 2001; and Chien et al., 2002; etc.*).

2.3.1 Analytic Method

2.3.1.1 Delay Models

Two well-known methods are widely used to analyze queuing delays caused by bottleneck: (1) the deterministic queuing models (*Abraham and Wang, 1981; Dudek and Rechard, 1982; Morales, 1986; Schonfeld and Chien, 1999*) and (2) the shock wave models (*Richard, 1956; Wirasinghe, 1978; Al-Deek et al., 1995; etc.*).

The deterministic queuing analysis is recommended by the Highway Capacity Manual (HCM) as the standard delay estimation technique for freeway zones (*TRB, 2000*). It is essentially a graphical procedure using a deterministic queuing diagram with the x-coordinate as time and the y-coordinate as the cumulative number of vehicles.

In the shockwave model, the traffic flow is assumed to behave like a fluid, and a backward shock wave develops when demand exceeds capacity. This model is often

used to estimate incident congestion. However, the shock wave speed is approximated based on traffic density, which is often difficult to measure or estimate.

2.3.1.2 Computerized Software

QUEWZ and QUICKZONE are the most used software packages for estimation of queue lengths and delays in work zones. Both of these software packages model traffic flow at a macroscopic level.

The computer model, called Queue and User Cost Evaluation of Work Zone (QUEWZ), was developed by Memmott and Dudek (*1984*) to assess work zone user costs. The most recent upgrade version is QUEWZ-98. It analyzes traffic conditions on a freeway segment with and without a lane closure in place and provides estimates of the additional road user costs and of the queuing resulting from a work zone lane closure. The road user costs calculated include travel time, vehicle operating costs, and excess emissions. That model does not consider any alternate path and the effect of diverting traffic to it.

QuickZone is a work zone delay estimation program developed in Microsoft Excel. The primary functions of QuickZone include quantification of corridor delay resulting from capacity decreases in work zones, identification of delay impacts of alternative project phasing plans, supporting tradeoff analyses between construction costs and delay costs, examination of impacts of construction staging, by location along mainline, time of day (peak vs. off-peak) or season, and assessment of travel demand measures and other delay mitigation strategies. The costs can be estimated for both an average day of work and for the whole life cycle of construction. The Maryland State Highway

Administration and the University of Maryland (*Kim and Lovell, 2001*) used QuickZone's open source code to customize the program to meet the State's needs. The University has added its own capacity estimation model to the program and has used a 24-hour traffic count, instead of the average daily traffic count found in original version. However, this program requires the users to input a great deal of information concerning a particular project, which may discourage the application of the software. To use the QuickZone program the user must first create a network of traffic facilities and then input hourly traffic volumes and capacities of all the links.

However, neither QUEWZ nor QuickZone has the function of optimizing work zone management plans.

2.3.2 Simulation

Although the concept of deterministic queuing model is widely accepted by practitioners for estimating queuing delay, the delay is usually underestimated (*Mcshane and Ross, 1992; Nam and Drew, 1998; Chien et. al., 2002; Najafi and Soares, 2001*) due to the neglected approaching and shock-wave delays. Besides, for a complex road network, analytical methods may not estimate user delays precisely.

As valuable analysis tools, microscopic traffic simulation models have been applied in various problems in work zone studies, such as the evaluation of traffic management plans, estimation of capacity and queue length, and optimization of traffic controls (*Nemeth and Rathi, 1985; Cohen and Clark, 1987; Chien and Chowdhury, 1998; Maze and Kamyab, 1999; Schrock and Maze, 2000; Lee, Kim and Harvey, 2005; etc.*).

CORSIM (including NETSIM and FRESIM), VISSIM, PARAMICS and INTERGRATION are among the most widely used microscopic simulation models.

Simulation models can output different measures of effectiveness (MOE's). In work zone analysis, delay, travel time, speed and volume are frequently used MOEs. However, simulation can be quite costly in terms of both computer and analysis time. Advanced computer and parallel processing techniques can be useful in decreasing the simulation time. The combination of analytic method and simulation method is also explored. Chien and Chowdhury (*2000*) developed a method to approximate delays by integrating limited amounts of simulation data and the concept of deterministic queuing model. In their study, simulation is applied to estimate average queuing delay with various ratios of entry volume to work zone capacity.

Only a few studies have been performed to date to validate the use of simulation models for work zone applications.

Dixon et al. (*1995*) evaluated the suitability of FRESIM for a simple freeway lane closure by comparing simulated behavior to the observed behavior of a study site. They concluded that FRESIM provided similar results to those observed in the field for low volume conditions. However, high volume conditions were not accurately simulated.

Middleton and Cooner (*1999*) evaluated three simulation models, CORSIM, FREQ and INTEGRATION, for simulating congested freeways. The calibration and validation performances of those models were tested using data collected on Dallas freeways. They concluded that all three models performed relatively well for uncongested conditions; however, the performance became sporadic and mostly unreliable for

congested conditions. Among those three models, CORSIM had the best overall performance, compared with the other two models.

Chitturi and Benekohal (*2003*) compared the queue length measured from field data to the results from FRESIM, QUEWZ, and QuickZone Software. They found that the results generated by QUEWZ did not match the field data. FRESIM either underestimated or overestimated the queue lengths. QuickZone generally underestimated the queue lengths generally.

2.4 Maintenance Decision Optimization

2.4.1 Long-term Maintenance Decision Optimization

Optimization based on mathematical programming models have been commonly used as aids in long-term project-level maintenance decision-making for single-year or multiyear prioritization. A variety of variables affecting the performance, safety, service, and cost of roadway sections can be optimized with or without considering various constraints such as acceptable condition constraint and budget constraint. One of the most popular issues is the problem of determining the optimal intensity (e.g. overlay thickness) and frequency (timing) of the pavement maintenance and rehabilitation (M&R) activities.

Mohammadi et al. (*1995*) optimized time intervals between M&R activities with the object of maximizing the value index similar to benefit/cost ratio. In this study deterioration model is simplified as a linear function. A simple enumeration method is applied to search the optimal solution. A similar methodology is introduced in NCHRP Report 523 (*2004*) for determining the optimal timing for applying preventive

maintenance treatments to highway pavement based on a deterministic parabolic deterioration model. The best timing scenario providing the largest B/C ratio is achieved by comparing several candidate time scenarios. However, it is unrealistic to use enumeration or comparison of all feasible solutions to achieve a global optimal solution especially when variable domains are continuous (e.g. timing). Many researchers formulated the optimization of M&R profile and timing as an optimal control problem and solved it by dynamic programming (DP) (*Fernandez and Friesz, 1981; Markow and Balta, 1985; Markow et al. 1993; Tsunokawa and Schofer, 1994; Mamlouk et al., 2000*). The optimal control formulation results in a complex combinatorial optimization problem, which is generally known to be computationally intractable. The DP algorithm can achieve optimal solutions but it suffers from rigidity and inability to handle large-scale problems. Li and Madanat (*2002*) formulated the problem of optimizing the frequency and intensity of pavement resurfacing under steady-state conditions for the case of deterministic Markovian deterioration and rehabilitation-only policy. They concluded that the optimal strategy is to undertake resurfacing upon reaching state-based[1] thresholds, which is the same as using time-based[2] thresholds when the deterioration pattern is deterministic. Their study removes the need to solve the problem as an optimal control problem. Although their objective functions are easy to understand, it is hard to express the solution in a simple analytical form. Thus they proposed to use a numerical way to solve optimization problem by trial and error. Tsunokawa et al. (*2006*) presented the use of two numerical optimization methods, the steepest gradient algorithm and the conjugate gradient

[1] State-based thresholds: The M&R activities are undertaken when a threshold value of level of service is reached.

[2] Time-based thresholds: The M&R activities are undertaken after pre-defined periods of infrastructure usage.

method, with what-if models to find the optimal schedule of maintenance works for a given road section. In the field of project-level pavement optimization, heuristic algorithms such as Genetic Algorithms are proved to be efficacious approaches to obtain near-optimal solutions within enormous possible solutions in sufficiently quick time spans (*Bosurgi and Trifiro, 2005; Elbehairy et al, 2006*). A Genetic Algorithm was also successfully used to optimize the maintenance scheduling of highway infrastructure elements, such as load signs, guardrails and luminaries, with budget constraint (*Maji and Jha, 2007*).

The optimal maintenance and rehabilitation problem taking into account the uncertainty in the measurement of facility condition or the deterioration process has been addressed and solved by various researchers (*Golabi et al., 1982; Markow and Balta, 1985; Carnahan, 1988; Madanat and Ben-Akiva, 1994; Tsunokawa and Schofer, 1994; Durango and Madanat, 2002; Sanchez-Silva et al., 2005*). Studies of incorporating both network-level and project-level decisions were also conducted. (*Liu et al. 1997; Miyamoto, 2001; Hegazy et al., 2004, Elbehairy et al., 2006*). Detailed review of relevant studies is not presented here since this dissertation concentrates on optimization of project-level decisions based on deterministic deterioration models.

2.4.2 Short-term Work Zone Decision Optimization

Considerable efforts have been devoted to optimizing work zone decisions to minimize negative impacts, usually measured by the total cost. McCoy et al. (*1980*) provided a simple framework to find the optimum work zone length by minimizing the total cost including construction, user delay, vehicle operating, and crash cost in construction and maintenance zones of rural four-lane divided highways. The user delay costs were

modeled based on average daily traffic (ADT) volumes, while the crash costs were computed by assuming that the crash rate per vehicle mile was constant in a work zone area. The optimal work zone length was derived based on 1979 data. Because the unit cost factors had changed considerably since 1981, McCoy and Peterson (*1987*) found the optimum work zone lengths to be about 64% longer that those used previously.

Martinelli and Xu (*1996*) added the vehicle queue delay costs into McCoy's (*1980*) model. The work zone length was optimized by minimizing the total user cost, excluding the maintenance and crash costs. Viera-Colon (*1999*) developed a similar model of four-lane highways which considered the effect of different traffic conditions and an alternate path. However, that study did not develop alternative selection guidelines for different traffic flows or road characteristics. Schonfeld and Chien (*1999*) developed a mathematical model to optimize the work zone lengths plus associated traffic control for two-lane, two-way highways where one lane at a time is closed under steady traffic inflows. They found the optimal work zone length and cycle time for traffic control and minimized the total cost, including agency cost and user delay cost. No alternative routes were considered in that study. They (*2001*) then developed another model to optimize the work zone length on four-lane highways using a single-lane closure strategy. Based on the previous work, Chen and Schonfeld (*2005a, 2005b*) developed work zone length optimization models for two-lane and four-lane highway with a single alternate route under steady traffic inflows.

Fwa, Cheu, and Muntasir (*1998*) developed a traffic delay model and used genetic algorithms to optimize scheduling of maintenance work for minimizing traffic delays

subject to several constraints on maintenance operations. Pavement sections, work teams, start time and end time for each section were scheduled. Other conditions in that study were given, e.g. work zone configuration and available work duration for each team, and road section length. These variables were not optimized in that study. Chang, Sawaya, and Ziliaskopoulos (*2001*) used traffic assignment approaches to evaluate the traffic delay, which include the impact of work zone combinations on an urban street network. A tabu search methodology was employed to select the schedule with the least network traffic delays.

Chien, Tang, and Schonfeld (*2002*) developed a model to optimize the scheduling of work zone activities associated with traffic control for two-lane two-way highways where one lane at a time is closed considering time-varying traffic volumes during four periods in a day: morning peak, daytime, evening peak, and nighttime periods. A greedy method is used as the search approach. Jiang and Adeli (*2003*) used neural networks and simulated annealing to optimize work zone lengths and starting times for short-term freeway work zones using average hourly traffic data, considering factors such as darkness and numbers of lanes. More complete scheduling plans for multiple-zone maintenance projects were not attempted in that work. Chen and Schonfeld (*2004*) developed a set of models to simultaneously optimize the work zone length, scheduling, lane closure strategy and diversion fractions, using simulated annealing search algorithm. Two-lane and multiple-lane highways, single and multiple detours as well as steady and time-varying traffic volume, and time constraints (*2006*) are considered in their models.

Tien and Schonfeld (2006) considered the potential benefit of accelerating the maintenance work with additional cost in the work zone optimization by developing a continuous tradeoff function between work time and cost. This study seeks to jointly optimize the work zone length, work rates and traffic diversion fractions for steady inflow conditions. Whereas the continuous time-cost tradeoff function and steady traffic may not reflect the reality.

Based on their previous study, Chen and Schonfeld (2007) introduced a new dimension of pavement durability and thus analyzed preferred pavement thickness as well as the work zone scheduling in optimizing costs per year rather than one-time cost per resurfacing project. However, the effect of traffic load on the pavement life is ignored and user costs resulting from pavement deterioration are not considered in the life cycle cost analysis.

Tang and Chien (2008, 2010) improved their analytical models by incorporating a discrete work time-cost function and utilizing user-equilibrium assignments instead of the system optimization method to decide traffic diversion faction. A genetic Algorithm (GA) is applied to reach near-optimal solutions. Their model focuses on optimizing work zone scheduling and maintenance productivities while keeping other work zone decisions such as lane closure configuration uniform in each work zone, which may limit the model extendibility by introducing other critical decision variables to further reduce the work zone cost. In addition, like most of the previous work zone optimization models handling time-varying traffic flow, the problem size of the model depends highly on the number of time intervals (usually ranging from 15 minutes to 1

hour) in the analysis period. Thus it is questionable whether these models can be applied in longer maintenance projects which may last a couple of weeks or months.

All the above studies used macroscopic analytical methods (e.g. deterministic queuing analysis) to estimate user delays. However, analytical methods are based on some simplified assumptions which lead to the neglect of detailed representations. Therefore analytical methods may not be able to provide satisfactory solutions for complex transportation networks.

With the increasing development of computing technology, simulation-based optimization has received considerable attention. This process seeks to find the best value of some decision variables for a system where the performance is evaluated based on the output of a simulation model of this system (*Olafsson and Kim, 2002*).

Cheu et al. (*2004*) presented a hybrid methodology to schedule maintenance activities at various sites in a road network, using genetic algorithm (GA) as an optimization technique, coupled with a traffic-simulation model to estimate the total travel time of users. This study demonstrated the availability of simulation-based optimization technology in work zone problems, although it did not focus on work zone length and on the duration optimization problem.

2.5 Simulation-based Optimization

Simulation in combination with a numerical optimization method is an effective tool for analyzing difficult stochastic optimization problems, especially when decision variables are interdependent in complex ways. According to the comprehensive reviews of literature on simulation optimization and its applications in real-world

provided by Andrad´ottir (*1998*), Azadivar (*1999*), Fu (*1994*), Fu (*2002*), Fu, Glover, and April (*2005*), Henderson and Nelson (*2006*), the main approaches used to solve simulation optimization problems consist of:

- Ranking & selection according to statistical analysis;
- Response surface methodology based on approximations of the objective function that the simulation model represents by a numerical representation;
- Gradient-based approaches primarily targeting continuous solution space;
- Random search approaches primarily targeting discrete solution space;
- Sample path optimization approaches which take a sufficiently large amount of simulations, and then try to optimize the resulting estimates in a deterministic way;
- Model-based approaches, which are not dependent explicitly on any current set of solutions, but use a probability distribution on the space of solutions to provide an estimate of where the best solutions are located;
- Metaheuristics, such as simulated annealing, genetic algorithm.

For solving large-scale real-world problems metaheuristics currently dominate other approaches focusing on convergence proof, mathematical tractability and optimal solution, though the latter ones receive more attention by the academic world. The main reason is that metaheuristics are generally fast, robust, and generate near-optimal solutions good enough in practice. A new research trend is to combine the robustness of metaheuristics with certain statistical analysis techniques or the established methods for guaranteeing performance (*Olafsson and Kim, 2002, Lee et al., 2006*).

Although increasing attention is being paid to simulation-based optimization, it still faces great challenges. The key feature in simulation optimization is its stochastic nature. According to Ho (2000) and Chen et al. (2000), if simulation models are used for optimization purposes, it is necessary to settle for "good enough with high probability" instead of asking "the best for sure" for the objective function cannot be evaluated exactly. As addressed by Banks (2000), optimization via simulation adds an additional complication because the performance of a particular design must be estimated through multiple replications. In practice, this could mean that simulation-based optimization can be quite costly in terms of both computer and analysis time. Thus the critical issue in simulation optimization is balancing the tradeoff between the computational effort used in estimating the objective function and that used for exploration of the finite feasible solution space in order to achieve good solutions in acceptable time budget.

There are three main methods for reducing the computational burden of solving the optimization problem via simulation.

The first is by reducing the number of candidate solutions examined and evaluated by simulation. Efficient search algorithms with fast convergence rate would be quite useful. The commercial package OptQuest attempts to compensate for this by using a neural network metamodel to screen out candidates which may have poor performance (*Fu, 2002*). Maintaining partial or complete information on solutions previously encountered is another way to save the number of solutions evaluated by simulation (*Pichitlamken and Nelson, 2001*).

The second is by reducing the computational time required to evaluate each candidate solution, either through reducing the number of replications needed for each simulation run or by reducing the number of simulation runs required. An idea comes from not treating the simulation process as a black box but letting optimization get more intelligent feedback from the simulation, and vice-verse. For example, RISKOptimizer, a simulation optimization software, allows the optimization engine to preemptively terminate a simulation early, when it is clear that the completed simulation result would probably not be of value in the optimizer's calculations (*Boesel, 2001*). Fu (*2002*) argues that providing some measure of goodness for the metaheuristics and developing practical and effective implementation of algorithms with proven convergence properties can reduce the number of simulation replications needed to obtain precise estimates. Guikema et al. (*2004*) proposed an approach using ridge regression to approximate the results of the full simulation run for some candidate solution evaluations in order to reduce the number of runs of simulation.

The third is by distributing the computational tasks to multiple processors. Most efforts focus on decomposing the simulation models into smaller ones (*Lee, 2004; Xu and Sen, 2005*) or distributing complete simulation runs or other optimization steps (*Lagnana et al., 2006*). Luo et al. (*2000*) introduced a framework for combining the statistical efficiency of simulation optimization techniques with the effectiveness of parallel execution algorithms based on the Internet and web-based technologies.

2.6 Expected Research Contribution

After a review of the above studies, it appears that many methods, consisting of both analytic methods and simulation models, have been developed and applied in various problems in maintenance studies, including but not limited to the assessment of work zone impacts and the optimization of work zone decisions. Due to complex and combinatorial nature of the work zone optimization problem, heuristic algorithm was adopted as the dominant problem solving approach.

In previous researches, long-term maintenance decision optimization is separated from short-term maintenance decision optimizations. However, there are some short-term decisions that may affect long-term roadway performance. It would be valuable to consider jointly optimization of these two types of decisions.

In terms of short-term work zone decision optimization, the state-of-practice shows that the administrative agencies select the best plan from several alternatives (e.g. select a lane-closure period from weekday night closure, weekday off-peak daytime closure, weekend closure) instead of applying optimization technology. The state-of-art of the academic research focus on developing advanced optimization method, nevertheless, based on some simplified assumptions that may enlarge the gap between research and practice. Most of the previous studies are based on analytic methods, which may not be precise due to over-simplified assumptions especially in complex traffic network. Few of studies integrate simulation with optimization. A main barrier is that simulation is a very time-consuming way to evaluate the objective function in an optimization process.

In addition, lane closure schedules in most major maintenance projects are periodic time windows. However, previous studies on work zone optimization did not fully utilize this characteristic and as a result the problem size and complexity increases with the maximal allowable project duration. This may potentially reduce the efficiency and scalability of their proposed search algorithm.

Addressing the above three issues, this dissertation attempts to achieve the following major contributions:

(1) Improve the analytical model to more precisely estimate the work zone impacts.

(2) In addition to work zone schedule and configuration, traffic impact mitigation strategies, as key components of a typical work zone management plan, will be introduced in the optimization model. Some practical issues, such as periodic lane closure time windows and different traffic patterns in weekday and weekend, will be considered in the proposed model with the object of decreasing the gap between theory and practice as well as improving the flexibility and scalability of the optimization procedure.

(3) This dissertation will develop an efficient simulation-based optimization algorithm to solve an optimization problem in which simulation is applied to estimate work zone impacts. This approach will analytically pre-optimize the decision variables before performing a detailed evaluation with microscopic simulation. The application of parallel computing will also be exploited in order to speed up the overall optimization process.

(4) Taking account of short-term and long-term impacts caused by different work zone decisions, this dissertation will combine those critical work zone management decisions in a comprehensive optimization model.

Chapter 3 One-Time Work Zone Cost Model

One-time work zone costs deals with agency and user costs brought about by the establishment of work zones during major maintenance operations. In this chapter, models are developed to quantitatively measure one-time work zone costs attributable to work zones on a multiple-lane highway under time-varying traffic inflow. While work zone agency costs are formulated analytically, work zone user costs are estimated by two different methods: one is based on a detailed microscopic simulation model and the other is based on a simplified analytical model that accounts for the effect of randomness in arrival flows, shock-wave, and merging behavior. The analysis procedure is illustrated in Figure 3-1.

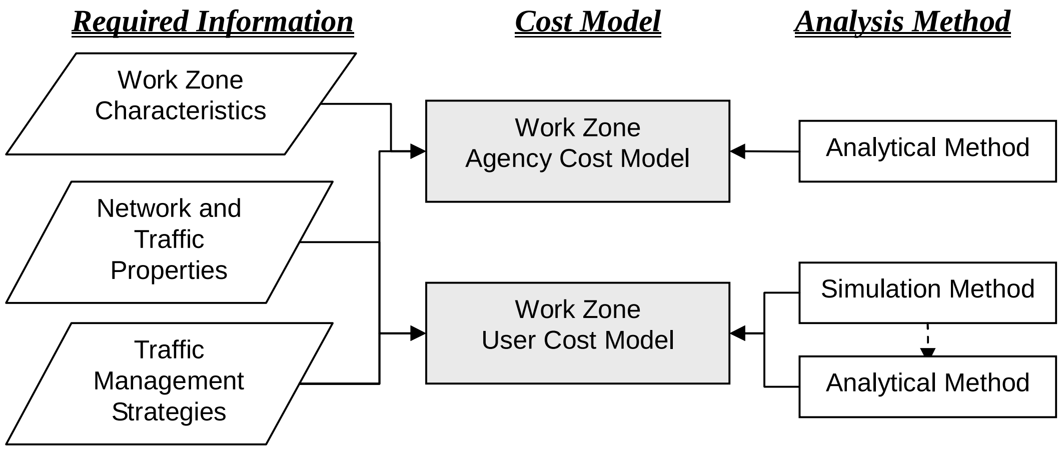


Figure 3-1 One-Time Work Zone Cost Analysis Procedure

3.1 Work Zone Characteristics

The Manual on Uniform Traffic Control Devices (MUTCD) (*FHWA, 2003*) refers to a work zone as a "temporary traffic control (TTC) zone" which generally consist of the advance warning area, the transition area, the activity area (including buffer space and work space), and the termination area. An example of a typical work zone configuration is shown in Figure 3-2. The characteristics used to describe a work zone

are categorized into three categories: work zone configuration, work zone timing and duration, and work productivity parameters.

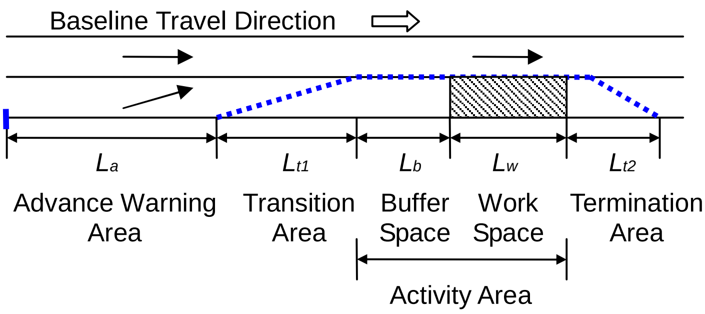


Figure 3-2 Single Lane-closure Work Zone Configuration

3.1.1 Work Zone Configuration

The relevant attributes defining the configuration of a work zone include the work zone length (L), the number of lanes closed for roadwork in the baseline direction (N), and the number of lanes occupied for contra-flow operation in the opposite direction (N').

(1) the work zone length (L)

This study defines the work zone length as the total length of the restricted section with physical space loss. The work zone length *(L)* can be calculated as the sum of the transition area length (L_{T1}), the buffer space length (L_B), the work space length (L_W), and the termination area length (l_{T2}), as shown in. Except the work space length (L_W), the lengths of the other work zone components (L_{T1}, L_B and L_{T2}) can be determined based on posted speed limit using the guidance provided in MUTCD (*FHWA, 2003*) and therefore they are considered as constant coefficients. The work zone length (L) is hence expressed in Eq. 3-1 as the sum of the work space length (L_w) and a fixed length required to set up a work zone ($L_f = L_{t1} + L_b + L_{t2}$):

$L= (L_{t1} + L_b + L_{t2}) + L_w = L_f + L_w$ Eq. 3-1

where, L = the total work zone length;

L_f = the fixed work zone setup length ($L_f = L_{t1} + L_b + L_{t2}$);

L_w = the work space length.

(2) the number of lanes occupied in both directions (N and N')

The roadway capacity reduction caused by a work zone mainly depends on the total number of lanes available and the number of lanes closed. Partial closure or full closure results in no disruption of traffic in the opposite direction. While the "contra-flow" concept is employed in lane closures by crossing over one or more lanes of traffic in the opposing direction to the bypass roadwork, the loss of capacity occurs in both directions. Figure 3-3 illustrates partial closure and crossover scenarios.

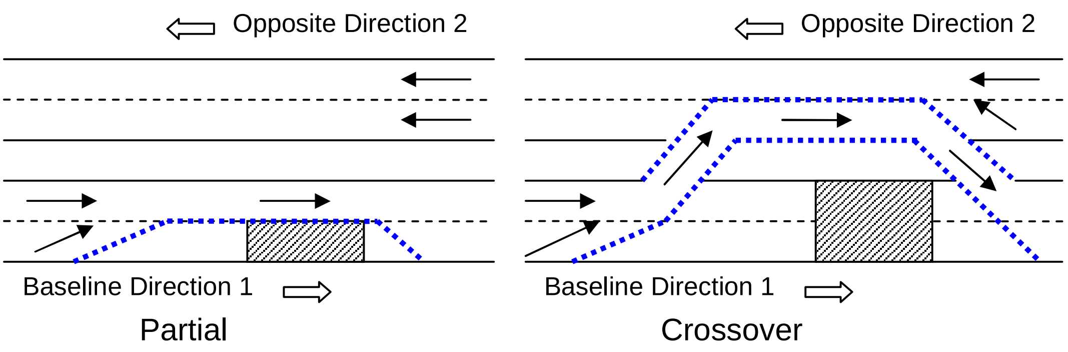


Figure 3-3 Two Lane Closure Types

The layout of the work zone lane closure is specified by the number of lanes closed in the normal travel direction (N) and the number of lanes occupied for contra-flow operation in the opposite direction (N_c). Given the number of lanes in two directions, N_1 and N_2, the number of remaining lanes available for travelling along the work zone section in both directions, N'_1 and N'_2, can be derived from the following equations:

$N'_1 = N_1 - N + N_c$ Eq. 3-2

$N'_2 = N_2 - N_c$ Eq. 3-3

where, N'_1 = the number of available lanes in the baseline direction 1 in work zone condition;

N'_2 = the number of available lanes in the opposite direction 2 in work zone condition;
N_1 = the total number of lanes in the baseline direction 1 in normal condition;
N_2 = the total number of lanes in the opposite direction 2 in normal condition;
N = the number of closed lanes in the baseline direction 1;
N_C= the number of usable contra-flow lanes in the opposite direction 2;

3.1.2 Work Zone Timing and Duration

As traffic demand varies over time, it is necessary to know the schedule of a work zone (time of day and day of week) and the duration of time a work activity which occupies the work zone. The time span of a work zone is determined by two parameters: the work starting time (S) and the work ending time (E), using *00::00* of the nearest Monday prior to the work starting as the reference time (*0*). An example of a work zone schedule is illustrated in Figure 3-4. The work zone during (D) can be derived from Eq. 3-4.

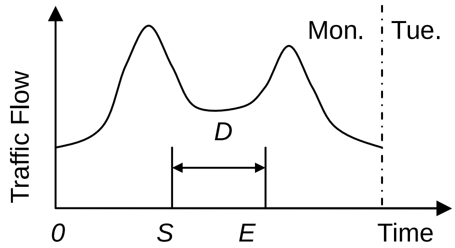


Figure 3-4 A Work Zone Schedule

$$D= E-S \qquad \text{Eq. 3-4}$$

3.1.3 Productivity Parameters

Four productivity parameters have to be specified to estimate the maintenance cost (C_M) and duration (D) of a work zone:

- The fixed setup cost per zone z_1;
- The average additional cost required per unit length per lane z_2;
- The fixed setup time per zone z_3;
- The average additional time required per unit length per lane z_4;

Here z_1 and z_3 denote the fixed cost and time needed for mobilization and demobilization purposes such as site preparation, cleaning-up, and traffic control setup while z_2 and z_4 denotes the additional cost and time required to complete maintenance

work for unit length. The values of these parameters depend on the work type (patching, grinding, resurfacing, etc.), construction method, deployed labor and equipment resources, etc. Considering that a wider work space may increase the operation efficiency and thereby reduce the unit work cost and time, efficiency factors, f_2 *and* f_4, are introduced to reflect this impact if additional lanes (N_A) are closed for providing access for demolition and construction activities.

Given the productivity parameters, the work zone maintenance cost and duration can be modeled as linear functions of the total length to be maintained in the work space, as expressed in the following equations:

$$C_M = z_1+z_2 \cdot (1+f_2 \cdot N_a) \cdot N_w \cdot L_w \quad \text{Eq. 3-5}$$

$$D = z_3+z_4 \cdot (1+f_4 N_a) \cdot N_w \cdot L_w \quad \text{Eq. 3-6}$$

where, L_w = the length of work space;

N_w = the number of maintained lanes;

N_a = the number of access lanes (N_a=N-N_w);

f_2 f_4 = the multi-lane operation efficiency factor;

3.2 Network and Traffic Information

The work zone traffic impact is directly dependent on the volume, distribution and operating characteristics of the traffic flows on corridors that are directly or indirectly impacted by the reduced service level of the restricted roadway section.

The road network to be analyzed should not only cover the mainline corridor in which traffic flow is directly affected by the work zone, but also contain the potential diversion routes, which are formed by the off-ramps exiting the mainline, the surrounding arterials or parallel highways, and the on-ramps to the mainline downstream the work zone section. It should be noted that a transportation agency normally demarcates only one detour route with guide signs (*Zhang et. al., 2008*).

Figure 3-5 illustrates a sample study network, which covers a mainline corridor in both directions (Q_1 and Q_2) and a detour route in both directions (Q_3 and Q_4).

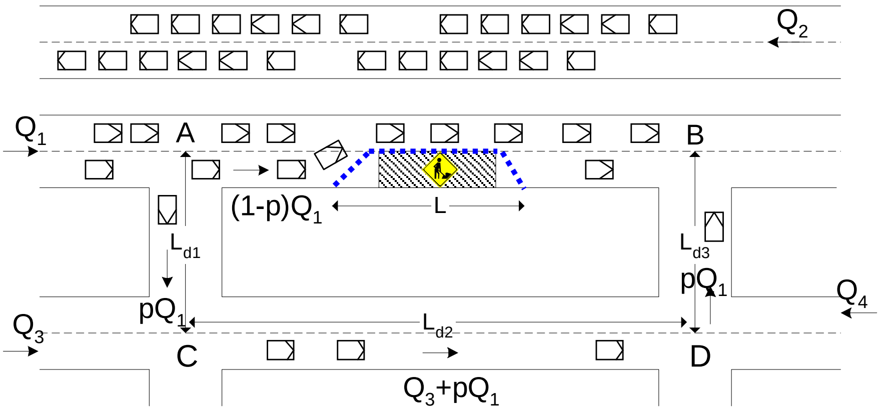


Figure 3-5 Study Network

Time-dependent traffic demand under normal conditions (without work zone) Q has to be provided to analyze the dynamic characteristics of the traffic during work zone operations. An hour is used as a duration unit in which traffic inflows stay appropriately constant. The 24-hour daily traffic distribution or the 168-hour weekly traffic demand should be provided, depending on whether there is significant distinction between weekday and weekend traffic hourly distributions. Since the operating costs and the value of time differ among vehicle types, the highway users are classified as passenger cars and heavy trucks. The proportion of heavy vehicle is denoted as P_T.

3.3 Traffic Management Strategies

With the booming development of information and Intelligent Transportation Systems (ITS) technologies, a variety of traffic management strategies have been widely applied to mitigate the work zone traffic impacts. Most of these strategies either focus on supply side by improving capacities on corridors that are directly or indirectly impacted

by work zones (supply-side) or try to influence travelers' response to the roadway restriction (demand-side). Therefore in this study a traffic management strategy is defined by the following five variables:

- The fixed employment cost per zone β_1;
- The average additional cost required per unit time β_2;
- Adjustment of the work zone capacity δ_w (%);
- Adjustment of the detour capacity δ_d (%);
- Adjustment of traffic diversion percentage δ_p (%);

3.3.1 Capacity Adjustment

Work zone capacity (C_w) is defined as the maximum number of vehicles passing the lane restriction over a one-hour period and it is dependent on a number of location-specific variables, such as lane closure configuration, work zone intensity and heavy traffic percentage. A baseline work zone capacity can be estimated using existing methods and computer programs (e.g. Highway Capacity Manual equation, University of Maryland Capacity Estimation equation, QUEWZ capacity model and simulation model) or through engineer judgment. In recent years several innovative work zone control strategies have been proposed to manage the traffic approaching and traveling through the work zone lane closures by means of speed control and merge control. Their benefit in terms of increased work zone throughput (capacity) and safety has been tested in simulation and field experiments. (*McCoy et al., 1999; Pesti et al., 1999; Walters et al., 2001; Tarko and Venugopal, 2001; Bearcher et al., 2004; Kang and Chang, 2006a; Kang and Chang, 2006b*).

The employment of capacity improvement strategies is not limited to work zones. The capacity of a detour route (C_d) can also be increased to attract more vehicles diverting from the mainline route as well as improve progression on alternative routes. Detour capacity can be improved through changing traffic signal timing plans, using reversible lanes, widening lanes, etc.

Given a set of capacity improvement strategies, the work zone capacity (c_w) and detour capacity (c_d) are changed to adjusted capacities (c'_w and c'_d) through the following relations:

$$c'_w = c_w + \Delta c_w [1 - \prod_{k=1}^{K} (1 - \delta_{w,k})] \quad \text{Eq. 3-7}$$

$$c'_d = c_d + \Delta c_d [1 - \prod_{k=1}^{K} (1 - \delta_{d,k})] \quad \text{Eq. 3-8}$$

where, c'_w, c'_d = adjusted work zone capacity and detour capacity;
c_w, c_d = baseline work zone capacity and detour capacity;
Δc_w, Δc_d = the maximal possible increase of capacities in work zone and detour;
$\delta_{w,k}$ = adjustment of work zone capacity contributing to the k^{th} strategy;
$\delta_{d,k}$ = adjustment of detour capacity contributing to the k^{th} strategy.

3.3.2 Demand Adjustment

When facing travel disruptions, vehicle operators who normally use the maintained roadway may change their travel behavior by diverting to alternative route, changing departure time, switching modes, or even canceling their trips. Unfortunately, the studies estimating how work zone characteristics and traffic management strategies affect the fractions of time shift, mode switch, and trip cancellation are far from well developed due to the complexity of human psychology and lack of real-world data. Therefore, this study only accounts for demand diversion in response to work zone

delay and agency guidance and assumes that O-D demand patterns remain the same as those under normal condition without work zones.

Detour strategies, such as Variable Message Signs (VMS) /Detour guide, advanced traveler information system, media camp, are desired to guide a percentage of traffic off the roadway under construction and onto other existing roadways. The purpose is to utilize spare capacity on the detour so that the traffic delay over the entire network can be reduced. Although traffic levels on the mainline are reduced, the diverted traffic would be delayed by taking a longer path and so as the original traffic traveling on the detours due to increased traffic density. The strategy can be beneficial only when the mainline delay saving outweighs the delay increase on the detour. It is essential to examine the ability of the detour routes to handle diverted traffic and analyze the potential network impacts evaluated by mainline user costs and detour user costs.

The diversion rate (*p*) is the percentage of vehicles diverted from their normal route during road constructions. In this study, whether to take travelers' route-changing behavior into account determines the way to model the impact of a detour strategy on the diversion ratio.

(1) Option 1: System Control

(1.1)User Input

Under the assumption that the performance of a detour strategy is controllable and predictable, the improvement of diversion rate δ_p is a user input, which is estimated from external analysis or through engineering judgment.

$$p' = p + (1-p)[1-\prod_{k=1}^{K}(1-\delta_{d,k})] \qquad \text{Eq. 3-9}$$

where, p' = adjusted diversion rate, $p' \in [0, 1]$;
p = natural diversion rate without any detour strategy, $p \in [0, 1]$;
$\delta_{d,k}$ = adjustment of diversion rate contributing to the k^{th} strategy, $\delta_{d,k} \in [0, 1]$.

(2.2) System Optimization Model

From a system control perspective, it is valuable to derive the optimal traffic diversion fraction based on the real-time conditions on the mainline route and alternate roads based on the assumption that the drivers cooperate with one another in order to minimize total system travel time. The optimal diversion rate is obtained by solving the following optimization problem:

$$\text{Min} \quad Z = x_w \cdot t_w(x_w) + x_d \cdot t_d(x_d + x_{ed}) + x_{ed} \cdot t_{ed}(x_d + x_{ed}) \qquad \text{Eq. 3-10}$$

Subject to $x_d + x_w = Q$; $x_d \geq 0$; $x_w \geq 0$

where, x_w = the remaining traffic flow on the original route;
x_d = the diverted traffic flow on the detour route;
x_{ed} = the existing traffic flow on the detour route;
t_w = the volume-dependent travel time on the original route;
t_d = the volume-dependent travel time on the detour route;
t_{ed} = the volume-dependent travel time on the major detour section;
Q = the traffic on the original route under normal condition.

(2) Option 2: User choice

In reality, the diversion fraction varies dynamically with the real-time traffic conditions and might be hard to predict in advance because network users are independent decision-makers in route choice. No one can force them to travel on a particular road. A considerable number of studies have indicated that a substantial percentage of drivers would divert while reliable travel time/delay information is provided by VMS or other traveler information systems. Based on field observations and survey results some quantitative methods have also been developed to help

engineers to determine diversion rates at work zones (*QUEWZ; QuickZone; Ullman and Dudek, 2003; Song and Yin, 2008; Zhang, et. al, 2008*) and they can be categorized into criteria-based algorithm, choice-based model and user equilibrium traffic assignment method.

(2.1) Choice-base Model

For short term or non-recurrent work zones, a binary logit model developed by Song and Yin (2008) is used to estimate the diversion fraction in response to en route detour strategies. Note that the model is intended for work zones on urban freeways where parallel frontage roads are available.

$$p' = p \qquad \text{if } t_w \le t_d \qquad \text{Eq. 3-11}$$

$$p' = 1 - (1-p)\frac{1}{1+\exp[0.1416(t_w - t_d) + 0.1054]} \qquad \text{if } t_w > t_d$$

where, p' = adjusted diversion rate, $p' \in [0, 1]$;
p = natural diversion rate without any detour strategy, $p \in [0, 1]$;
t_w = estimated travel time on the original route;
t_d = estimated travel time on the detour route;

(2.2) User Equilibrium Model

For long duration or recurrent work zones, travelers may quickly learn from their travel experience and adjust their route choice. In this situation, it is reasonable to assume user equilibrium can be reached and no road users can decrease their travel effort by unilaterally switching paths (*Daganzo and Sheffi, 1977*). The diversion is obtained by solving the following traffic assignment problem:

$$\text{Min} \qquad Z = \int_0^{x_w} t_w(x_w)dx_w + \int_0^{x_d} t_d(x_d + x_{ed})dx_d \qquad \text{Eq. 3-12}$$

Subject to $x_d + x_w = Q$; $x_d \ge 0$; $x_w \ge 0$
where, x_w = the remaining traffic flow on the original route;

x_d = the diverted traffic flow on the detour route;
x_{ed} = the existing traffic flow on the detour route;
Q = the traffic on the original route under normal condition.

3.4 One-Time Work Zone Cost Model

Alternative work zone management plans can be compared through their one-time work zone costs (C_T) if they have negligible or equal long-term impacts. Assuming that a roadway maintenance project is divided into m consecutive work zones and maintenance work is undertaken only on one work zone at a time, the one-time work zone total cost (C_T) is the summation of the agency cost ($C_{A,i}$) and user cost ($C_{U,i}$) of all the work zones, shown in Eq.3-13.

$$\begin{aligned} C_T &= C_A + C_U \\ &= \sum_{i=1}^{m} C_{A,i} + \sum_{i=1}^{m} C_{U,i} \\ &= \sum_{i=1}^{m} (C_{M,i} + C_{S,i} + C_{I,i}) + \sum_{i=1}^{m} (C_{D,i} + C_{V,i} + C_{E,i}) \end{aligned} \qquad \text{Eq.3-13}$$

where, $C_{A,i}$ = Agency Cost of the i^{th} work zone
$C_{U,i}$ = User Cost of the i^{th} work zone
$C_{M,i}$ = Agency Maintenance Cost of the i^{th} work zone
$C_{S,i}$ = Agency Traffic Mitigation Cost of the i^{th} work zone
$C_{I,i}$ = Agency Equipment/Labor Idling Cost of the i^{th} work zone
$C_{D,i}$ = User Delay Cost of the i^{th} work zone
$C_{V,i}$ = User Vehicle Operating Cost of the i^{th} work zone
$C_{E,i}$ = User Expected Accident Cost of the i^{th} work zone

3.4.1 Work Zone Agency Cost Model

Agency costs include all direct expenses associated with the placement of a maintenance treatment. These include maintenance cost, traffic management cost, and idling cost.

3.4.1.1 Maintenance Cost

Maintenance cost includes labor, equipment, material and administration costs spent on maintaining a work zone. It is formulated as a linear function of the maintained area with the form in Eq.3-14)

$$\begin{aligned} C_{M,i} &= z_1 + z_2 \cdot (1+f_2) \cdot N_{wi} \cdot L_{wi} \\ &= z_1 + z_2 \cdot (1+f_2) \cdot \frac{D_i - z_3}{z_4 \cdot (1+f_4)} \end{aligned} \qquad \text{Eq.3-14}$$

where, N_{wi} = the number of maintained lanes in the i^{th} work zone(#);
L_{wi} = the work space length of the i^{th} work zone (mile);
D_i = the duration of the i^{th} work zone (hr);
z_1 = the fixed setup cost per work zone ($/zone);
z_2 = the unit length maintenance cost ($/lane-mile);
z_3 = the fixed setup time per work zone (hr/zone);
z_4 = the unit length maintenance time (hr/lane-mile);
f_2 = the multi-lane operation cost saving factor (%);
f_4 = the multi-lane operation time saving factor (%).

3.4.1.2 Traffic Management Cost

Given K traffic management strategies available in the project, the cost spent on employing those strategies is formulated as a linear function of the work zone length in Eq.3-15:

$$C_{S,i} = \sum_{k=1}^{K} b_{k,i} (\beta_{1,k} + \beta_{2,k} D_i) \qquad \text{Eq.3-15}$$

where, $b_{k,i}$= dummy variable indicating whether the k^{th} strategy is implemented in the i^{th} work zone ($b_{k,i} = 1$ if the k^{th} strategy is activated in the i^{th} work zone , *if* $=0$,otherwise);
$\beta_{1,k}$= the fixed employment cost of the k^{th} strategy;
$\beta_{2,k}$= the unit-time cost of the k^{th} strategy;
K= total number of strategies;
D_i= the duration of the i^{th} work zone.

3.4.1.3 Idling Cost

It is assumed that crews and equipments are idle during the work break between two consecutive work zones. Being idle is usually an undesirable situation, since there is an opportunity cost of not earning returns on the idle asset. For that reason, idling cost is considered in work zone activities. It is modeled as the product of idling time and the average cost of idling crews and equipments v_I.

$$C_{I,i} = (S_{i+1} - E_i)v_I \qquad \textit{for } i=1,2,\ldots,I-1 \qquad \text{Eq.3-16}$$
$$C_{I,i} = 0 \qquad \textit{for } i=I$$

where, S_i = the starting time of the i^{th} work zone
E_i = the ending time of the i^{th} work zone

3.4.2 Work Zone User Cost Model

The impact of work zones on traffic is measured by the added user costs caused by the presence of work zones. They are the costs that motorists incur because of reduced travel speed, restricted capacity at work zones, and additional travel distance due to detours. Three major portions of added road user cost are delay cost, added vehicle operating cost and expected accident cost to highway users resulting from maintenance activities.

3.4.2.1 User Delay Cost

Delay costs should be determined using the amount and value of lost time resulting from delays caused by work zone activities. Note that delay estimation is not limited to vehicles traveling through the work zone area. Traffic movements across the detour routes should also be analyzed, if any natural or designated diversion phenomenon occurs.

$C_{D,i} = D_{D,i} \cdot v_D$ Eq.3-17

where, v_D = the weighted average of value of time delay costs for passenger cars and for trucks

$D_{D,,i}$ = the total delay caused by the i^{th} work zone.

The accuracy of user delay estimates significantly affects the measure of work zone management plan. If well calibrated, microscopic simulation models, which model each vehicle as a separate entity moving in a network, are usually expected to provide more accurate estimates of user delays compared to analytical procedures, especially when testing the effectiveness of various traffic management strategies or conducting network analysis. In this study, CORSIM (Corridor Simulator), a comprehensive microscopic traffic simulation program developed by the Federal Highway Administration (FHWA), is used to simulate work zone conditions and estimate the user delay.

However, application of microscopic simulation can be quite expensive in terms of data collection and computational time. To save time and effort while maintaining a desirable precision level, an analytical model derived from simulation analysis is developed to estimate work zone delays in a typical network shown in Figure 3-5. To consider all the users affected by the work zones, the total delay caused by the i^{th} work zone ($D_{D,i}$) consists of the delay of the mainline flow, the delay of original flow on detour incurred by diverted traffic and the delay of detoured flow, as shown in Eq.3-18.

$$D_{D,i} = D_i^{m1} + D_i^{m2} + D_i^{d} + D_i^{p} \qquad \text{Eq.3-18}$$

where, D_i^{m1}, D_i^{m2} = the delay of the mainline flow in directions 1 and 2;

D_i^{d} = the delay of the original flow on detour route;

D_i^{p} = the delay of detoured flow.

To more accurately estimate mainline delay (D_i^{m1} and D_i^{m2}), which is directly caused by work zone bottlenecks, a new term of systematic delay is introduced into the analytical

delay estimation model in addition to the deceleration, moving, acceleration, and queuing delay (Eq.3-19 and Eq.3-20). Systematic delay accounts for the effect of the stochastic nature of traffic flows and other delays that are difficult to model analytically. Its formulation is derived from CORSIM simulation results.

$$D_i^{m1} = D_{d,i}^{m1} + D_{m,i}^{m1} + D_{a,i}^{m1} + D_{q,i}^{m1} + D_{r,i}^{m1} \qquad \text{Eq.3-19}$$

$$D_i^{m2} = D_{d,i}^{m2} + D_{m,i}^{m2} + D_{a,i}^{m2} + D_{q,i}^{m2} + D_{r,i}^{m2} \qquad \text{Eq.3-20}$$

where, $D_{d,i}^{m1}, D_{d,i}^{m2}$ = the deceleration delay occur in directions 1 and 2 along mainline;
$D_{m,i}^{m1}, D_{m,i}^{m2}$ = the moving delay occur in directions 1 and 2 along mainline;
$D_{a,i}^{m1}, D_{a,i}^{m2}$ = the acceleration delay occur in directions 1 and 2 along mainline;
$D_{q,i}^{m1}, D_{q,i}^{m2}$ = the queuing delay occur in directions 1 and 2 along mainline;
$D_{r,i}^{m1}, D_{r,i}^{m2}$ = the systematic delay occur in directions 1 and 2 along mainline.

The details of the simulation and analytical delay estimation procedures are presented in the next subsections.

3.4.2.2 Vehicle Operating Cost

Changes in speed due to slower design speeds and decreased capacity of the facility will have an effect on vehicle operating costs (VOC) as a result of excess consumptions of fuel and oil, maintenance, and tires.

A CORSIM Measures of Effectiveness (MOE) related to vehicle operating cost is the fuel consumed for seven classes of vehicles in gallons or miles per gallon. The fuel consumption cost can be obtained by multiplying the prevailing fuel price. Although CORSIM maintains tabulated data for fuel consumption rate which is expressed as a function of acceleration, given the vehicle performance index and vehicle speed, these data are difficult to collect and calibrate and have not been updated within CORSIM in

many years. As indicated in a guideline provided by FHWA, these data are recommended for comparison analysis only (*FHWA, 2007*).

Work zone VOC can also be calculated analytically by summing up three major VOC components: speed change cycle VOC, queue idling VOC, and detour VOC, denoted as $C_{Vs,i}$, $C_{Vq,i}$, and $C_{Vd,i}$, respectively.

$$C_{V,i} = C_{Vs,i} + C_{Vq,i} + C_{Vd,i} \qquad \text{Eq.3-21}$$

Speed change cycle VOC is the additional vehicle operating cost associated with decelerating from the unrestricted upstream approach speed to the work zone speed and then accelerating back to the unrestricted approach speed from the work zone speed after traversing the work zone. The excess cost of speed change cycles is calculated as the production of the number of mainline vehicles affected by work zone *i* and unit cost per speed change cycle v_s (\$/cycle):

$$C_{Vs,i} = v_s \cdot \int_{S_i}^{E_i} [(1 - p'(t)) \cdot Q_1(t) + b_i^c \cdot Q_2(t)]dt \qquad \text{Eq.3-22}$$

where, $Q_1(t)$ = the traffic volume in direction 1 along mainline;
$Q_2(t)$ = the traffic volume in direction 2 along mainline;
$p'(t)$ = the adjusted diversion rate;
b_i^c = a dummy variable indicating whether lane closure type is crossover.

Queue idling VOC is the additional vehicle operating costs associated with "stop and go" driving in the queue where traffic operates under "Forced Flow" conditions. This operating cost, denoted as $C_{Vq,i}$, can be formulated as the product of work zone queuing delay (veh.hr) and unit vehicle idling cost v_q (\$/veh.hr):

$$C_{Vq,i} = v_q \cdot [D_{q,i}^{m1} + D_{q,i}^{m2}] \qquad \text{Eq.3-23}$$

Excess operating costs are due to travel longer distance along detours. This vehicle operating cost is the cost per vehicle-mile times the volume taking the detour times the additional distance traveled on the detour compared with the work zone route:

$$C_{Vd,i} = v_d \cdot (L_d - L_m) \int_{S_i}^{E_i} p(t) \cdot Q_1(t) dt \qquad \text{Eq.3-24}$$

where, v_q = average operating cost per unit distance ($/veh.mile);
L_d = the travel distance along mainline route (mile);
L_m = the travel distance along detour route (mile);

3.4.2.3 Expected Accident Cost

Many studies have found that crash rates are significantly higher in work zones than they are on the road section under normal operations. The impact is aggravated when the congestion level is high. Simulation models assume "100-percent safe driving" so CORSIM is not effective in predicting the change in expected accident rate.

The accident cost incurred by the traffic passing the work zone, is determined from the number of crashes per 100 million vehicle hours of travel (γ_E) multiplied by the product of the total delay ($D_{D,i}$) and the average cost per crash (v_E) (*McCoy and Peterson, 1987, Pigman and Agent, 1990; Chien and Schonfeld, 2001*).

$$C_{E,i} = D_{D,i} \cdot \gamma_E \cdot v_E \qquad \text{Eq.3-25}$$

3.5 Delay Estimation based on Simulation Method

CORSIM is a powerful tool to evaluate pre-specified work zone operations based on detailed representations of traffic characteristics, network geometry characteristics, work zone characteristics and traffic control plans.

3.5.1 Introduction to CORSIM

Traffic simulation models can be classified into microscopic, macroscopic or mesoscopic. Microscopic models address and describe the movement of each individual vehicle in the traffic flow independence of the movement of the adjacent vehicles, both in the longitudinal (car-following behaviour) and in the lateral (lane-changing behaviour) sense. Macroscopic models describe the traffic flow as a fluid with particular characteristics via the aggregate traffic variables traffic density, flow, and mean speed. Mesoscopic models track individual vehicles but group them into platoons with same behaviors, and thus provide the precise level in the middle of microscopic and macroscopic simulation models.

CORSIM is a microscopic and stochastic simulator. It represents single vehicles entering the road network at random times moving second-by-second according to local interaction rules that describe governing phenomena such as car following logic, lane changing, response to traffic control devices, and turning at intersections according to prescribed probabilities.

CORSIM combines two of the most widely used traffic simulation models, NETSIM for surface streets, and FRESIM for freeways. CORSIM simulates traffic and traffic control systems using commonly accepted vehicle and driver behavior models and it has great ability to model complex road networks, various traffic conditions and different traffic control alternatives. CORSIM can handle networks of up to 500 nodes and 1,000 links containing up to 20,000 vehicles at one time.

For reflecting the realistic traffic operations and predicting traffic performance correctly, the CORSIM simulation model has to be properly calibrated by fine tuning the parameters discussed in the next subsection.

3.5.2 Work Zone Simulation in CORSIM

For the CORSIM model, the input data specified by the user consists of a sequence of "record types", which contains a specific set of data items as well as an identification number. These data specified in a "record type" are called "entries".

In NETSIM and FRESIM, different record types are used to contain work zone related information.

3.5.2.1 NETSIM

(1) Record Type 11

The record type 11 is the NETSIM link description, which describes the geometry and the traffic characteristics of NETSIM links.

The entries 11-17 specify the channelization for all defined lanes. We can simulate a link with one or more closed lanes, by setting proper values of the channelization codes for corresponding lanes. A closed lane can be treated as a transient condition that is due to a construction zone. The entries 23 and 24 specify the mean startup delay and the mean queue discharge headway (in tenths of a second), which may affect signalized intersection capacity in a NETSIM link.

However, only full lanes can be channelized and the capacity of the whole link can be changed. If we want to simulate a work zone segment within a surface street link, we have to divide the link into several links. Also, the road capacity along work zone segment and drivers' behavior characteristics are still hard to calibrate in NETSIM. Therefore, in this study we focus on freeway work zones.

(2) Record Type 21

Turn movement data for surface street links are recorded in the record type 21. These data will change when detours are used.

3.5.2.2 FRESIM

(1) Record Type 29

A comprehensive freeway incident simulation procedure is provided in FRESIM. It is recommended by the user manual for work zone modeling. The user can specify either blockages or "rubbernecking" to occur on a lane-specify basis. The rubbernecking factor (in a percentage) represents the reduction in capacity for vehicles in remaining open lanes in the work zone area. Each incident occurs at the specified longitudinal position on a freeway link, extends over the user-specified length of the roadway, and last for any desired length of time (CORSIM Users' Manual).

With the above function, it is convenient to set up: (a) number of closed lanes; (b) location of work zone (left, center or right of the road); (c) work zone length; (d) starting time of the work zone; (e) work zone duration; (f) location of the upstream warning sign for a work zone; (g) rubbernecking factor in the remaining open lanes in

the work zone area. FRESIM is therefore attractive for simulating work zone conditions due to this freeway incident specification function, which is defined in the record type 29.

(2) Record Type 20

Record Type 20 is used to record freeway link operation data. In this record type, the information contributing to work zone operation includes:

- Desired free-flow speed in a freeway link

This parameter specifies the desired, unimpeded, mean free-flow speed (in miles per hour) that is attained by traffic, in the absence of any impedance due to other vehicles, control devices or work zone activities. Under work zone conditions, the speed limit and driver's compliance behavior in the freeway link may be changed. Then the free-flow speed of the work zone link and the upstream links may need to be reset.

- Car-following sensitivity multiplier in a freeway link

The car-following sensitivity multiplier permits users to adjust the car-following sensitivity on a link-by-link basis in a FRESIM network. The car-following sensitivity factor represents a driver's desire to follow the preceding car. The value of car-following sensitivity multiplier in a link contributes to the vehicle capacity of this link.

(3) Record Type 25

Turn movement data for freeway links are recorded in the record type 25. These data will be updated when detours are used.

3.5.3 MOEs in CORSIM

After the test network is built in CORISM based on input data including work zone characteristics, network and traffic information, and traffic management strategies, it is essential to match the simulation model with field data to ensure the reliability and quality of the simulation results. For freeway work zones, the rubbernecking factor, car-following sensitivity factor, and desired free-flow speed on work zone link and upstream links are key parameters to adjust in calibration process.

CORSIM output file consists of cumulative NETSIM link statistic data, NETSIM movement specific Statistics, cumulative FRESIM link statistics, FRESIM network statistics and network-wide average statistics for each time period. Since we intend to evaluate the work zone effect from the system point of view, the "Total Time (vehicle-hours)" and "Total Delay (vehicle-hours)" in network-wide average statistics are major MOEs used in delay estimation. The net work zone delay (D_D) is the difference between the network-wide total time with work zone (TT_w) and that without work zone (TT_{wo}).

$$D_D = TT_w - TT_{wo} \qquad \text{Eq.3-26}$$

CORSIM can estimate the total fuel consumed by all vehicles of the specified type on each link. Two environmental MOEs, "FRESIM cumulative values of fuel consumption (sub-network data)" and "NETSIM cumulative values of fuel consumption (sub-network data)", can be used to calculate additional fuel consumption due to work zone:

$$F = \sum_{a=1}^{A} (F_{a,w}^{FRESIM} + F_{a,w}^{NETSIM}) - \sum_{a=1}^{A} (F_{a,wo}^{FRESIM} + F_{a,wo}^{NETSIM}) \qquad \text{Eq.3-27}$$

where, F = additional fuel consumption due to work zone;

A = total type of vehicle class (A=7 in CORSIM);

$F_{a,w}^{FRESIM}$ = fuel consumption of the a^{th} type of vehicles in FRESIM under work zone condition;

$F_{a,w}^{NETSIM}$ = fuel consumption of the a^{th} type of vehicles in NETSIM under work zone condition;

$F_{a,wo}^{FRESIM}$ = fuel consumption of the a^{th} type of vehicles in FRESIM under normal condition;

$F_{a,wo}^{NETSIM}$ = fuel consumption of the a^{th} type of vehicles in NETSIM under normal condition;

In order to reduce the statistical variance in simulation analysis, multiple simulation replications must be run with different random number seeds. The running time of each simulation run depends on scope of the size of the network, the number of time periods, and traffic congestion level.

3.5.4 Traffic Flow Properties

In a macroscopic traffic flow model, speed is derived from the relation among flow, speed and density, while in a microscopic simulation model speed is derived from the car following theory. This section discusses the properties of steady-state traffic flow based on car following models embedded in CORSIM, in particular the associated speed-flow relations.

3.5.4.1 Car-Following Model in CORSIM

The basic car-following logic incorporated in CORSIM is that vehicles attempt to maintain constant space headway between the lead and follower vehicles. Under steady-state conditions, this logic can revert to the Pipes car-following model (*Rakha and Crowther, 2002*). In Pipes' model, the distance headway has a linear relation with speed. The car-following behavior of a vehicle is constrained by a maximum speed, which is commonly known as the free flow speed.

FRESIM uses the Pitt car-following model developed by the University of Pittsburgh to determine the follower vehicle acceleration (*Halati et al., 1997*). The logic is as follows:

$$H = H_j + kV + bk\Delta V^2 = H_j + kV \qquad \text{Eq.3-28}$$

Under steady-state conditions $\Delta V=0$

where, H = distance headway between lead and follower vehicles ;
H_j = jam density headway;
k = driver sensitivity factor for the follower vehicle;
b = calibration constant which equals 0.1 if the speed of the follower vehicle exceeds the speed of the lead vehicle, otherwise it is set to 0;
V = speed of the follower vehicle;
ΔV = difference in speed between lead and follower vehicle
V_f = free-flow speed;

The car-following model in NETSIM incorporates a driver reaction time and the ability of vehicles to decelerate at feasible rates without resulting in vehicle collisions (*Rakha and Crowther, 2002*).

$$H = H_j + \Delta S + \Delta R + S_F - S_L = H_j + V \cdot \Delta t = H_j + \frac{1}{3600}V \qquad \text{Eq.3-29}$$

Under steady-state conditions,
$\Delta V=0$, $\Delta S=V\Delta t$, $\Delta R=0$, $S_F=S_L$, $\Delta t=1\text{second}=1/3600$ *hour*

where, ΔS = distance traveled by follower vehicle over time interval Δt
ΔR = distance traveled by follower vehicle during its reaction time
S_F = distance traveled by follower vehicle to come to a complete stop
S_L = distance traveled by lead vehicle to come to a complete stop

Either in Eq.3-28 or in

Eq.3-29, distance headway has a linear relation with speed. Note that the vehicle speed (V) is constrained by a maximum speed, which is commonly known as the free flow speed (V_f).

$$V = \min(\frac{H - H_j}{k}, V_f) \qquad \text{Eq.3-30}$$

3.5.4.2 Conversion to Traffic Flow Model

The car-following model of traffic has a harmonious tie-in to macroscopic theory. The following procedures can integrate the two approaches.

The macroscopic traffic flow models identify the relation between the three traffic flow parameters, namely flow (*Q*), speed (*V*), and density (*K*), which can be measured fairly easily in the field using standard loop detectors or traffic counters.

$$Q = KV \qquad \text{Eq.3-31}$$

Assuming all vehicles in the traffic stream maintain the same headway distance, we obtain the following relationship between distance headway (*H*) and density (*K*):

$$H = 1/K \qquad \text{Eq.3-32}$$

By substituting Eq.3-32 into Eq.3-30 and Eq.3-31, the microscopic car-following model can be related mathematically to the macroscopic speed-density relationship through the following forms:

$$V = \min(\frac{H - H_j}{k}, V_f) = \min(\frac{1/K - 1/K_j}{k}, V_f) \qquad \text{Eq.3-33}$$

$$Q = KV = \frac{V}{H} = \frac{V}{H_j + kV} = \frac{V}{1/K_j + kV} \qquad \text{Eq.3-34}$$

A numerical test is conducted to compare the obtained traffic stream model and Greenshields' traffic flow model, one of the best known traffic stream models. In Greenshields' model, speed is a linear function of density, given in Eq.3-35.

$$V = V_f - \frac{V_f}{K_j} K \qquad \text{Eq.3-35}$$

$$Q = KV = \frac{K_j}{V_f}(V_f - V)V = K_j V - \frac{K_j}{V_f} V^2 \qquad \text{Eq.3-36}$$

The relations among flow (Q), density (K), and speed (V) in the two models are illustrated in Figure 3-6, given K_j=80 veh/mile, V_f=80 mph, k=3/6400, and road capacity=1600 vph. Unlike single-regime Greenshield's model, the traffic flow model converted from CORSIM car-following logic is multi-regime in the sense that a different model is utilized for the congested versus uncongested regimes. Specifically, traffic stream speed is insensitive to the traffic density in the uncongested regime.

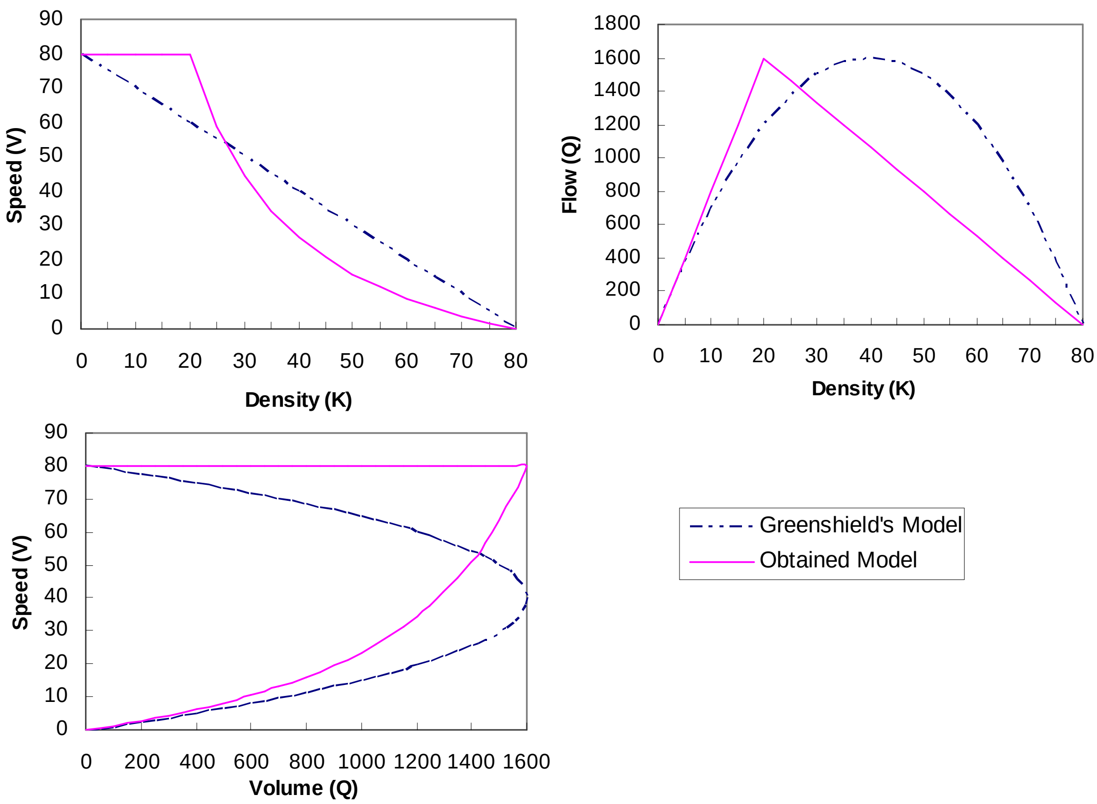


Figure 3-6 Comparison of obtained traffic flow model and Greenshield's Model

3.5.5 A Simulation Experiment

To investigate work zone delay and speed-flow relationship in work zone link, a simulation experiment is conducted based on a freeway segment in Maryland on the U.S. Route I-83 south bound with a right-lane closure work zone near the overpass bridge of Cold Bottom Road (Figure 3-7). The original free flow speed on I-83SB is 65 mph. It was modeled with CORSIM as a unidirectional two-lane freeway segment consisting of upstream links, one work zone link, and downstream links. An incident is modeled in the work zone link for replicating the one-lane closure area.

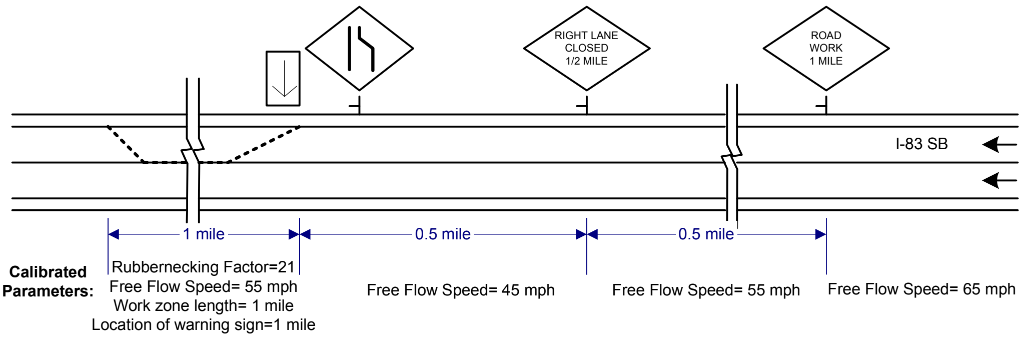


Figure 3-7 Test Work Zone Site and Calibrated Simulation Parameters

We use the field results as a baseline and try to calibrate the simulation model based on the observation results. The model calibration with respect to the upstream volumes, truck percentage, work zone throughput, and average speed at merge point was based on the field data collected in 2003. Table 3-1 demonstrates the calibration results.

Table 3-1 Calibration Results for the CORSIM Simulation Network

Traffic conditions	Actual data (10/10/2003)	Simulation results	
		Before calibration	After calibration
Upstream volume (2 lanes)	1875 vph	1875 vph	1875 vph
Heavy truck percentage	19 %	19 %	19 %
Average speed at merge point*	22.0 mph	13 mph	21 mph
Work Zone throughput	1,340 vphpl	1,646 vphpl	1,338 vphpl

Note(*): Merge point is located ahead the first work zone tape.

24 scenarios are run with volume to work zone capacity ratio ranging from 0.1 to 2.0 with an increment of 0.1. Each simulation lasts 9,000 seconds including a 3,600 second period with specified inflow and a 5,400 second period with zero inflow to clear vehicles traveling in the network. The initialization time is forced to 1,200 seconds. Each MOE is the average of results from 10 independent simulation replications with different random number seeds. Table 3-2 lists the average delay per vehicle (min/veh) and the average speed in the work zone link (mph) for each scenario.

Table 3-2 Simulation Results for 24 Scenarios (I-83SB)

Scenario No.	V/C Ratio	Volume (vph)	Capacity (vph)	Work Zone Delay (min/veh)	Work Zone Speed (mph)	Scenario No.	V/C Ratio	Volume (vph)	Capacity (vph)	Work Zone Delay (min/veh)	Work Zone Speed (mph)
1	0.10	134	1338	0.55	53.94	13	1.05	1405	1338	3.82	42.01
2	0.20	268	1338	0.56	53.16	14	1.10	1472	1338	5.35	42.43
3	0.30	401	1338	0.59	52.61	15	1.15	1539	1338	7.13	42.56
4	0.40	535	1338	0.72	51.67	16	1.20	1606	1338	8.87	42.88
5	0.50	669	1338	0.78	50.96	17	1.30	1739	1338	12.77	43.00
6	0.60	803	1338	0.88	49.72	18	1.40	1873	1338	16.27	43.18
7	0.70	937	1338	0.94	48.66	19	1.50	2007	1338	20.54	43.15
8	0.80	1070	1338	1.15	46.58	20	1.60	2141	1338	24.17	43.16
9	0.85	1137	1338	1.27	45.33	21	1.70	2275	1338	27.75	43.16
10	0.90	1204	1338	1.43	44.00	22	1.80	2408	1338	32.08	43.09
11	0.95	1271	1338	1.58	42.96	23	1.90	2542	1338	35.87	43.08
12	1.00	1338	1338	2.14	42.28	24	2.00	2676	1338	39.90	43.14

Figure 3-8 shows how the average work zone delay and work zone speed change with the traffic congestion level. From the simulation results and model animation, it can be seen that:

(1) Under uncongested conditions (V/C <1), most vehicles can easily find acceptable gaps to merge into the open lane without disturbing the traffic flows. Moving through the work zone area with lower speed is the major cause of delay. As the traffic volume increases, the work zone delay increases slowly and the speed in open lane decreases gradually in a near linear fashion.

(2) Under congested conditions (V/C≥1), vehicles begin to experience difficulties in changing lanes and consequently cause traffic disturbance. Queues form in both lanes at the upstream point of the blockage link, which leads to dramatic raise of work zone delay. In this queuing situation the work zone link operates at full capacity and the vehicles travel in the open lane with a relatively stable speed (43 mph) slightly lower than work zone speed limit (55 mph). This result shows the multi-regime property of the traffic stream modeled in CORSIM.

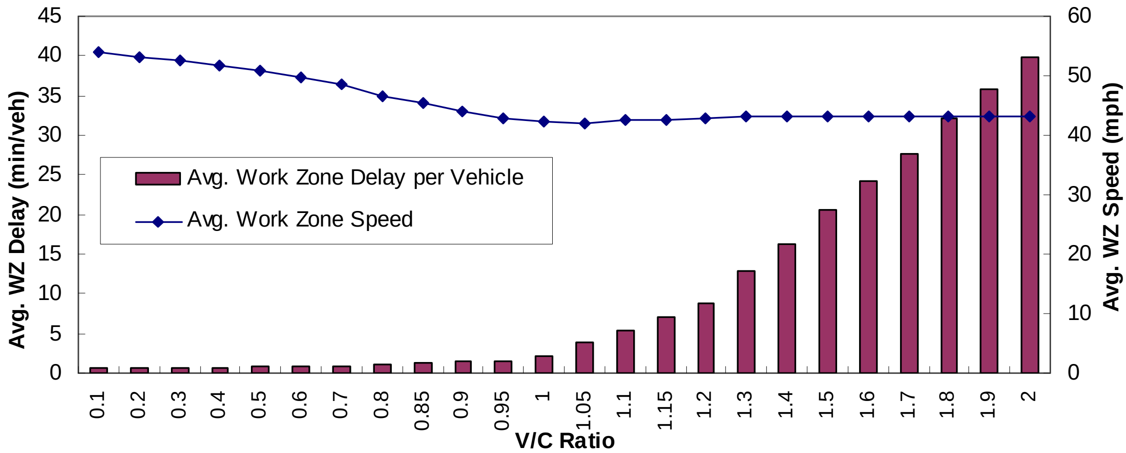


Figure 3-8 Change of Average Delay and Speed with Congestion Level

3.5.6 Strength and Limitations

A well-calibrated CORSIM simulation model is flexible to model complex work zone projects and observe the traffic impacts on the entire network, not just near the work zones. It has ability to model unusual geometric or traffic control features that are not handled in traditional methodologies. Its function of creating a real-time traffic animation allows analysts to observe vehicle interactions and visualize the potential

results of alternative scenarios. It must be noted that there are also some limitations that may cause difficulties in work zone simulation.

(1) Simulation time

CORSIM can simulate up to 19 time periods with a maximum duration of 9999 seconds in each time period. Thus, the total simulation time cannot exceed 52.7 hours. Hence, we cannot simulate a work zone whose duration exceeds 52.7 hours in one CORSIM input file (TRF file). Since CORSIM is incapable of exporting or importing network status, dividing one scenario into multiple input files may sacrifice accuracy.

(2) Incident properties in record type 29

For record type 29, which is used to simulate freeway work zones, CORSIM only allows users to specify the onset time of an incident, which is measured from the start of the simulation, at up to 9999 seconds. This indicates that the start time of the simulation has to occur less than 9999 seconds ahead of the work zone starting time and two zones cannot be successive in a TRF file if the first zone's duration exceeds 9999 seconds.

In addition, the duration of an incident cannot exceed 99999 seconds and the length affected by the incident cannot exceed 99999 feet, which limits on the duration and length of the work zone simulated in CORSIM.

(3) Vehicles entering the study network

We noted that CORSIM has difficulty dealing with storage of vehicles on short, congested links. Once the queues extend back to the entrance node and block vehicles from entering the network at their scheduled time, vehicles that were

scheduled to depart were not able to do so. The "departure delays" of those vehicles backed up behind entrance nodes will not be included in the total delay estimates in output statistics. This limitation may result in underestimating user delays in over-saturated conditions if upstream links cannot provide enough queue storage space.

Based on the above discussion, we can see CORSIM is a powerful tool for analyzing work zone mobility impacts over a large geographic area during certain time period (e.g. a.m or p.m. peak hours). It is suitable for studying multiple concurrent and potentially interacting work zone projects in a network or testing the performance of implementing different work zone traffic management strategies. However, CORSIM may not be able to accurately analyze the combined impacts of multiple work zones in consecutive time slots over a long period of time (e.g. several days or weeks).

3.6 Delay Estimation Based on Analytic Method

To save time and effort while maintaining a desirable precision level, an analytic model adjusted from simulation analysis is developed to estimate work zone delays in a typical network shown in Figure 3-5, where a single detour (Direction 3) is designated for mainline traffic (Direction 1). This model provides analytical formulations for calculating the delays experienced by mainline and detour users due to the i^{th} work zone under time-varying traffic demands.

3.6.1 Model Assumptions

The following assumptions are made in formulating the work zone delay estimation model:

(1) The maintenance work in i^{th} work zone (i = 1, 2, …., I) is conducted within time slot [S_i, E_i] over J_i duration units d_{ij} (j =1, 2, …., J_i), in which inflows stay appropriately constant, as shown in Figure 3-9.

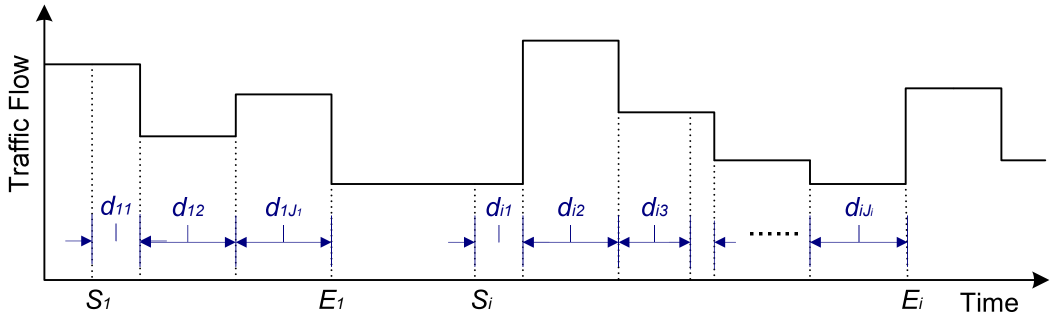


Figure 3-9 Delay Terms at Work Zone Area

(2) Mainline traffic demand can detour in response to work zone delay and agency guidance. Time shift, mode switch, and trip cancellation are not taken into account in this study. The O-D demand patterns remain the same as those under normal condition without work zones. The diverted fraction is estimated based on the criteria presented in Section 3.3.2 "Demand Adjustment".

(3) The travel times on the original and detour routes are estimated from BPR function. Possible signal or stop sign delays on the detour are considered. Queue backups to the mainline road are neglected.

(4) It is assumed K types of traffic management strategies are implemented in all work zones.

(5) Under the normal situation without a work zone, roadway capacity always exceeds traffic demand on both mainline and detour routes.

3.6.2 Mainline Delay

3.6.2.1 Mainline Capacity and Traffic Inflow

Given baseline work zone capacity c_w, work zone configuration and the information of traffic management strategies, the available capacities in Directions 1 and 2 during time period d_{ij} with the i^{th} work zone, denoted as $c'_{w1}(i,j)$ and $c'_{w2}(i,j)$, are product of the number of open lanes and the adjusted work zone capacity per lane, which can be obtained from Eq. 3-2 Eq. 3-3 and Eq. 3-7.

(1) The roadway capacity in Direction 1:

$$c'_{w1}(i,j) = c'_w N'_1 = \{c_w + \Delta c_w [1 - \prod_{k=1}^{K} (1 - \delta_{w,k})]\}(N_1 - N_i + N'_i) \quad \text{Eq.3-37}$$

(2) The roadway capacity in Direction 2:

$$c'_{w2}(i,j) = c'_w N'_2 = \{c_w + \Delta c_w [1 - \prod_{k=1}^{K} (1 - \delta_{w,k})]\}(N_2 - N'_i) \quad \text{Eq.3-38}$$

(3) The adjusted traffic inflows in Direction 1 is calculated based on diversion rate *p(i,j)*:

$$Q'_1(i,j) = Q_1(i,j)[1 - p'(i,j)] \quad \text{Eq.3-39}$$

(4) The traffic inflows in Direction 2 is not affected by work zone operations:

$$Q'_2(i,j) = Q_2(i,j) \quad \text{Eq.3-40}$$

3.6.2.2 Mainline Delay Formulation

The delay incurred by the restricted capacity at work zones (D_D) includes initial deceleration delay (D_d), queue move-up time (D_q), moving delay (D_m) and final acceleration delay (D_a). Figure 3-10 illustrates the components of work zone mainline delay. Four deterministic analytical equations are used to approximate these four delay

components and a model derived from simulation results is developed to estimate systematic delay (D_r), which accounts for stochastic nature of traffic flows.

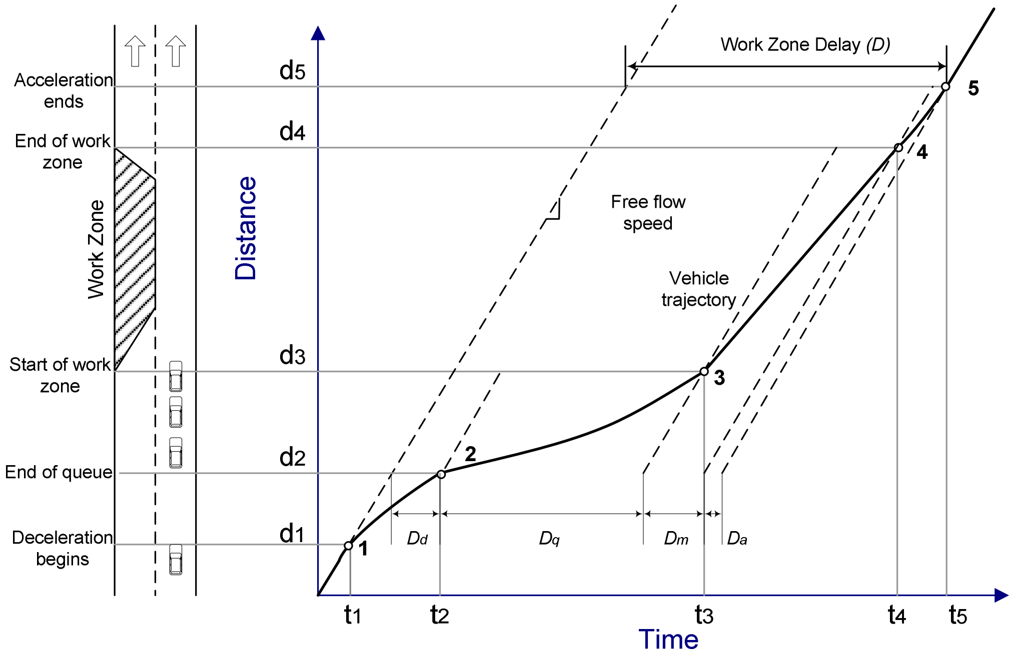


Figure 3-10 Delay Terms at Work Zone Area

(1) Deceleration Delay

Deceleration delay is defined as the difference between the time used by a vehicle to gradually reduce its speed from the free-flow speed (v_f) to the work zone speed (v_w) over a deceleration distance (s_d) and the free-flow time associated with that distance. The deceleration distance is assumed to be fixed and it is determined as the distance between the first advanced warning sign and the first tube of the work zone (e.g. 1 mile). The equation for deceleration delay of duration d_{ij} is obtained as:

$$D_d(i, j) = s_d\left(\frac{2}{v_f + v_w(i, j)} - \frac{1}{v_f}\right)Q'(i, j)d_{ij} \quad \text{Eq.3-41}$$

Base on the simulation results shown in Figure 3-8, a flow-speed model is developed to estimate average work zone speed of hour d_{ij} in Eq.3-42. As shown in Figure 3-11, work zone speed is assumed to linearly decrease from work zone speed limit (v_{wf}) to a full-capacity speed (v_{wq}) with increasing congestion level.

$$v_w(i,j) = \begin{cases} v_{wf} - \dfrac{Q'(i,j)}{c'_w(i,j)}(v_{wf} - v_{wq}) & \textit{Under uncongested condition} \\ v_{wq} & \textit{Under congested condition} \end{cases} \quad \text{Eq.3-42}$$

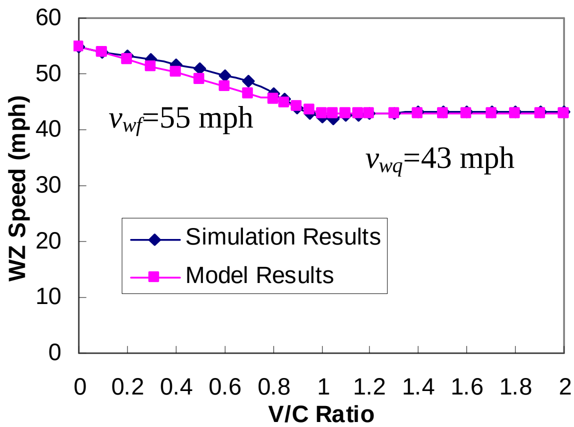


Figure 3-11 Work Zone Speed Model

(2) Queuing Delay

Since upstream demand may exceed work zone capacity and the whole system cannot reach steady state, stochastic queuing theory is not applicable to analyze work zone queuing delay. Therefore, the deterministic queuing model illustrated in Figure 3-12 is used to estimate the queue length $q(i,j)$ at the end of duration d_{ij} and queuing delay $D_q(i,j)$.

Within duration d_{ij}, if the inflow $Q'(i,j)$ exceeds the capacity $c'_w(i,j)$ a queue grows. Otherwise, the existing queue decreases. The cumulative number of vehicles in a queue at the end of d_{ij} is:

$$q(i,j) = \max\{0, q(i,j-1) + [Q'(i,j) - c'_w(i,j)]d_{ij}\}$$ Eq.3-43

The queuing delay of the hour d_{ij}, represented by $D_q(i,j)$, is obtained as

$$D_q(i,j) = \frac{q(i,j-1) + q(i,j)}{2} d_{ij}$$ Eq.3-44

If the queues have not dissipated yet when the work zone ends at the time of E_i, the queuing delay during an additional queue dissipation time $T_{dq,i}$ should be taken into account. Therefore the analysis period for the i^{th} work zone should be extended to E'_i determined in Eq.3-45. Note that the queue may be unable to dissipate completely at the beginning of next work zone ($i+1$) if the break time between two work zones is not long enough.

$$E'_i = \min\{E_i + T_{dq,i}, S_{i+1}\}$$ Eq.3-45

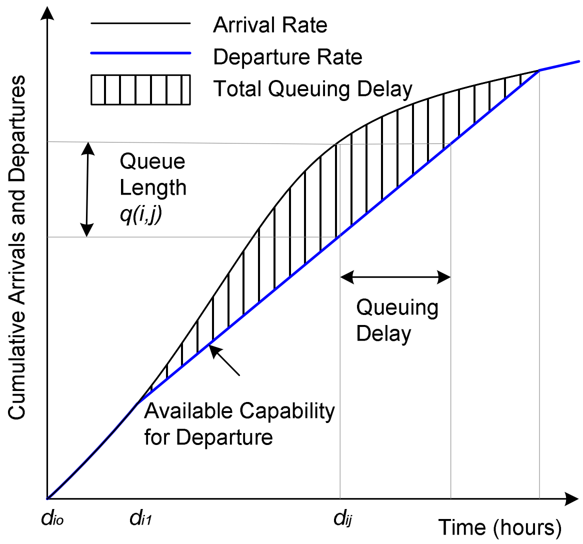


Figure 3-12 Deterministic Queuing Model

(3) Moving Delay

The moving delay $D_m(i,j)$ is obtained by multiplying the outflow passing through work zone area (L_i) by the difference between the travel time on the road with and without a work zone. The total volume is the sum of inflow $Q(i,j)$ and queue length accumulated from the previous duration $q(i,j-1)$ if they can be discharged within d_{ij}. Otherwise, the total volume passing the work zone area is the capacity volume $c'_w(i,j)$.

$$D_m(i,j)=\begin{cases}[\frac{1}{v_w(i,j)}-\frac{1}{v_f}]L_i[Q'(i,j)+q(i,j-1)]d_{ij} & if \quad q(i,j)=0 \\ [\frac{1}{v_w(i,j)}-\frac{1}{v_f}]L_i c'_w(i,j)d_{ij} & if \quad q(i,j)>0\end{cases} \tag{Eq.3-46}$$

(4) Acceleration Delay

Assuming a constant acceleration rate (a_a) (e.g. 2.5 m/s^2, *Shibuya et.al.,1999*), the delay incurred while the outflow accelerating from the work zone speed (v_w) to gain full operating speed (v_f) is derived as:

$$D_a(i,j)=\begin{cases}\frac{[v_f-v_w(i,j)]^2}{2a_a v_f}[Q'(i,j)+q(i,j-1)]d_{ij} & if \quad q(i,j)=0 \\ \frac{[v_f-v_w(i,j)]^2}{2a_a v_f}c'_w(i,j)d_{ij} & if \quad q(i,j)>0\end{cases} \tag{Eq.3-47}$$

(5) Systematic Delay

The above mathematical equations used to estimate deceleration, moving, acceleration, and queuing delays are developed under deterministic assumption. They do not consider stochastic vehicle arrivals and interaction among the vehicles. After applying the deterministic analytic models for the 24 scenarios in I-83SB case, we found that they always underestimate the overall magnitude of delays compared to stochastic

simulation analysis. The comparison of results of two methods is illustrated in Table 3-3 and Figure 3-13.

Table 3-3 Comparison of Results of Simulation Model and Deterministic Analytic Model

Scenario No. (#)	V/C Ratio	**Simulation Results** **Avg. Delay** (min/veh)	**Deterministic Analytical Model Results** *Dd* (min/veh)	*Dq* (min/veh)	*Dm* (min/veh)	*Da* (min/veh)	**Avg. Delay** (min/veh)	**Difference** **Avg. Delay** (min/veh)
1	0.10	0.55	0.09	0.00	0.15	0.00	0.24	0.31
2	0.20	0.56	0.10	0.00	0.16	0.00	0.27	0.29
3	0.30	0.59	0.11	0.00	0.19	0.00	0.30	0.29
4	0.40	0.72	0.12	0.00	0.21	0.01	0.33	0.39
5	0.50	0.78	0.13	0.00	0.23	0.01	0.36	0.41
6	0.60	0.88	0.14	0.00	0.25	0.01	0.40	0.48
7	0.70	0.94	0.15	0.00	0.28	0.01	0.44	0.50
8	0.80	1.15	0.16	0.00	0.30	0.01	0.47	0.67
9	0.85	1.27	0.17	0.00	0.32	0.01	0.49	0.77
10	0.90	1.43	0.18	0.00	0.33	0.01	0.51	0.92
11	0.95	1.58	0.18	0.00	0.34	0.01	0.54	1.04
12	1.00	2.14	0.19	0.00	0.36	0.01	0.56	1.58
13	1.05	3.82	0.19	1.50	0.36	0.01	2.06	1.76
14	1.10	5.35	0.19	3.00	0.36	0.01	3.56	1.79
15	1.15	7.13	0.19	4.50	0.36	0.01	5.06	2.07
16	1.20	8.87	0.19	6.00	0.36	0.01	6.56	2.31
17	1.30	12.77	0.19	9.00	0.36	0.01	9.56	3.21
18	1.40	16.27	0.19	12.00	0.36	0.01	12.56	3.72
19	1.50	20.54	0.19	15.00	0.36	0.01	15.56	4.98
20	1.60	24.17	0.19	18.00	0.36	0.01	18.56	5.61
21	1.70	27.75	0.19	21.00	0.36	0.01	21.56	6.19
22	1.80	32.08	0.19	24.00	0.36	0.01	24.56	7.52
23	1.90	35.87	0.19	27.00	0.36	0.01	27.56	8.31
24	2.00	39.90	0.19	30.00	0.36	0.01	30.56	9.34

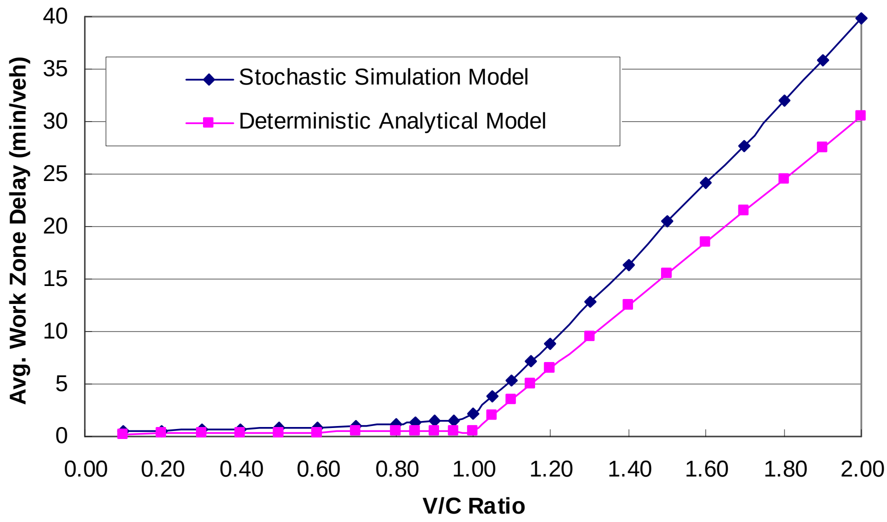


Figure 3-13 Average Work Zone Delay Obtained from Two Methods

To take account of the effect of the stochastic nature of traffic flows and other delays that are difficult to model analytically (e.g. shockwave delay, merging delay), an additional delay term called systematic delay is introduced into the analytic model.

Assuming that the difference between average work zone delays obtained from two methods is the observed systematic delay, a closed-form regression model shown in Eq.3-48 is developed to estimate average systematic delay per vehicle $t_r(i,j)$. Observed and predicted systematic delays are compared in Figure 3-14. The total systematic delay is calculated by multiplying average systematic delay per vehicle with the incoming traffic flow during d_{ij}.

$$t_r(i,j) = 0.098 + 0.279x + 1.143x^3 \quad \textit{(minute/vehicle)} \qquad \text{Eq.3-48}$$
$$= 0.0016 + 0.0047x + 0.0191x^3 \quad \textit{(hour/vehicle)}$$

$$D_r(i,j) = t_r(i,j) \cdot Q'(i,j) d_{ij} \qquad \text{Eq.3-49}$$

where, $x = Q'(i,j)/C'_w(i,j)$ (V/C Ratio);
$t_r(i,j)$ = average systematic delay per vehicle during d_{ij};
$D_r(i,j)$ = total systematic delay during d_{ij};

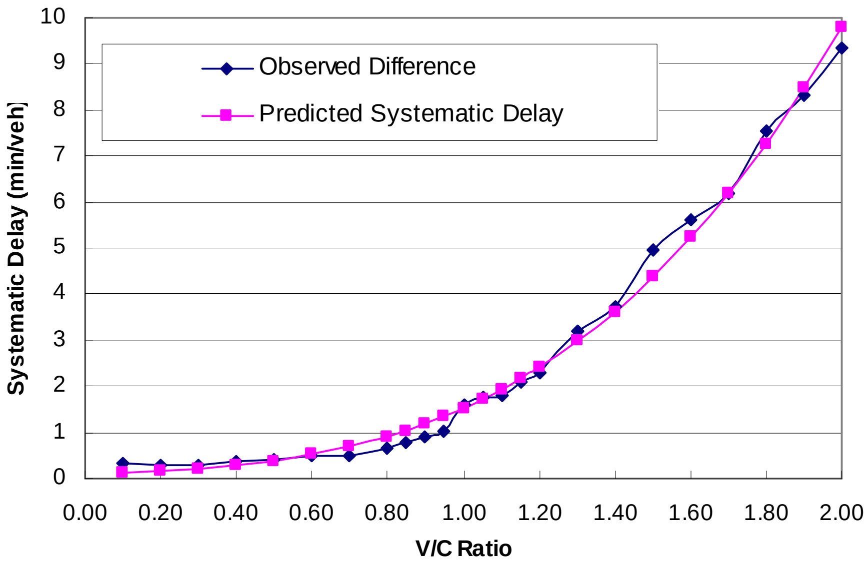


Figure 3-14 Predicted and Observed Systematic Delay

3.6.3 Detour Delay

(1) Diverted Flow on the Detour

The travel time of the diverted flows may increase for traveling a longer path with a

lower speed than if they stayed on the original path in normal condition. If the traffic is diverted from a highway or freeway to signalized arterials, additional queuing/stop delays due to intersections or stop signs should also be considered. When a detour is used, the moving delay of the traffic flow which is diverted from the mainline to the detour can be calculated with the following equations:

$$D^p(i,j) = (t_d(i,j) - t_{mf})Q_1(i,j)p'(i,j)d_{ij} \qquad \text{Eq.3-50}$$

$$= \left(\frac{L_{AC}}{v_{AC}} + \frac{L_{CD}}{v'_{CD}(i,j)} + \frac{L_{DB}}{v_{DB}} - \frac{L_{AB}}{v_f}\right)Q_1(i,j)p'(i,j)d_{ij}$$

where, $t_d(i,j)$ = the travel time along detour route during d_{ij};
t_{mf} = the travel time along mainline route in normal condition;
L_{AC} = the travel distance from the mainline exit to parallel detour entrance;
L_{CD} = the length of the parallel detour;
L_{DB} = the travel distance from the detour exit to mainline entrance;
L_{AB} = the length of mainline path;
v_{AC} = the average speed on L_{AC};
$v'_{CD}(i,j)$ = the speed on L_{CD} in duration d_{ij};
v_{DB} = the average speed on L_{DB}
v_f = the free-flow speed on the original path without work zones.

The updated Bureau of Public Roads (BPR) function proposed by Skabardonis and Dowling (*1997*) is used to relate changes in travel speed to increases in travel volume on detour route. This updated BPR-type model produces better fit to real-world data and simulation results than standard BPR function especially when volume-to-capacity ratios exceed 1.0.

$$v'_{CD}(i,j) = \frac{v_{CD}}{1 + a(x)^b} = \frac{v_{CD}}{1 + a\left[\dfrac{Q_1(i,j)p'(i,j) + Q_3(i,j) + q_3(i,j)}{c_{CD}}\right]^b} \qquad a=0.20,\ b=10 \qquad \text{Eq.3-51}$$

where, v_{CD} = the adjusted free-flow speed on detour route;
c_{CD} = the capacity of the detour route;
x = Volume/Capacity ratio;
a = coefficient in BPR fuction;
b = exponent in BPR fuction.

To consider the effects of signalization on arterial detour routes, the free-flow speed v_{CD} should be adjusted to include delay due to the presence of signals under low-volume conditions.

$$v_{CD} = \frac{L_{CD}}{L_{CD} / v_{mCD} + n_s \cdot w_s} \tag{Eq.3-52}$$

where, L_{CD} = length of the arterial detour route;
v_{mCD} = midlock (speed-limit) free-flow speed on detour route;
n_s = the number of intersections and stop signs along detour route;
w_s = the average waiting time passing intersections and stop signs along detour route.

(2) Original Flow on the Detour

The original vehicles on the diversion route would also increase their travel time because an increase in traffic will result in slower flow. When a detour strategy is applied, the delay of the original flow on the detour, as affected by the diverted flow, can be obtained from the following equation:

$$D^d(i,j) = (t_d(i,j) - t_{df})Q_3(i,j)d_{ij} = \left(\frac{L_{CD}}{v'_{CD}(i,j)} - \frac{L_{CD}}{v_{CD}}\right)Q_3(i,j)d_{ij} \tag{Eq.3-53}$$

where, $t_d(i,j)$ = the travel time along detour route during d_{ij};
t_{df} = the original travel time along detour route in normal condition;

3.7 Traffic Diversion Model

As discussed in Section 3.3.2 "Demand Adjustment", the diversion rate can be entered as an input parameter, and it can also be a result of an embedded diversion module in which diversion rate hinges on the difference between the travel time on the mainline route and the detour route. This section presents three approaches to obtain the diversion rate (p') during time period d_{ij}, depending on how travelers' route-changing behavior is taken into account (System Optimization assignment, Discrete Choice

Model and, and User Equilibrium assignment). The travel time estimation model is based on the following simplifications:

- The diverted fraction is constant within each time period unit d_{ij};
- Upstream volumes on mainline and detour routes are known;
- A natural diversion rate is pre-specified. Its default value is zero;
- The diverted traffic volume will not exceed the capacities of the road exiting and re-entering the mainline route (e.g. off-ramp and on-ramp). Therefore, a maximal allowed diverted volume $Q_{p,max}$ is given as an input parameter;
- It is assumed that the traffic flow will pass the work zone area with current speed $v_w(i,j-1)$ and that the traffic will regain the free flow speed after passing the work zone area.
- The travel time increases due to the excessive demand on mainline and detour routes are estimated through the updated Bureau of Public Roads (BPR) function (*Skabardonis and Dowling, 1997*);
- The travel speeds on the links connecting mainline and alternative routes are considered as constant.

(1) Travel Times on Mainline and Detour

The travel time on the mainline, denoted by $t_m(i,j)$ is estimated by summing up the travel times passing the section upstream of the bottleneck, the work zone section, and the section downstream of the work zone. The travel time on the detour route includes the time accessing the parallel alternative road, the time spent on the alternative road,

and the time going back to the mainline. The mainline and detour travel times can be obtained from Eq.3-54, Eq.3-55 and Eq.3-56.

$$t_m(i,j) = f_1(p') \qquad \text{Eq.3-54}$$

$$= \frac{L_{mu}}{v_f}[1 + a_1\left(\frac{Q_1(i,j)(1-p'(i,j)) + q_1(i,j-1)}{c'_w(i)}\right)^{b_1}] + \frac{L_i}{v_w(i,j-1)} + \frac{L_{md}}{v_f}$$

where, L_{mu} = length of the mainline section upstream of the work zone;
L_i = length of the work zone section;
L_{md} = length of the mainline section downstream of the work zone;
V_f = free flow speed on mainline;
V_w = work zone speed on mainline;
Q_1 = original traffic flow using mainline route;
q_1 = the number of vehicles waiting in an existing queue;
a_1, b_1 = updated BPR function parameters for mainline links;

$$t_d(i,j) = f_2(p') = \frac{L_{AC}}{v_{AC}} + \frac{L_{DB}}{v_{DB}} + t_{CD}(i,j) \qquad \text{Eq.3-55}$$

$$t_{CD}(i,j) = f_3(p') \qquad \text{Eq.3-56}$$

$$= \frac{L_{CD}}{v'_{CD}(i,j)} = \frac{L_{CD}}{v_{CD}} \cdot \left[1 + a_3 \cdot \left(\frac{Q_1(i,j)p'(i,j) + Q_3(i,j) + q_3(i,j-1)}{c_{CD}}\right)^{b_3}\right]$$

where, L_{AC} = length of the link connecting the exit ramp and the alternative road;
L_{CD} = length of the alternative road;
L_{DB} = length of the link connecting the alternative road and the entry ramp;
V_{AC},,V_{CD},V_{DB} = free flow speed on links AC, CD, and DB;
c_{CD} = capacity of the alternative road;
Q_1, Q_3 = original traffic flow using mainline route and alternative road;
q_3 = the number of vehicles waiting in an existing queue on the alternative road;
a_3, b_3 = updated BPR function parameters for the alternative road;

Once the predicted travel time on mainline and detour routes, $t_w(i,j)$ and $t_d(i,j)$, are obtained, the actual diversion rate $p'(i,j)$ can be calculated from the choice-based model or user equilibrium model presented in Section 3.3.2.

(2) Traffic Assignment Models

System Optimization Assignment

$$Min \quad Z(p') = Q_1(1-p')f_1(p') + Q_1 p' f_2(p') + Q_3 f_3(p') \qquad \text{Eq.3-57}$$

User Equilibrium Assignment

$$Min \quad Z(p') = \left| f_1(p') - f_2(p') \right| \qquad \text{Eq.3-58}$$

Choice-based Model

$$p' = p \qquad \text{if } f_1(p) \le f_2(p) \qquad \text{Eq.3-59}$$

$$p' = 1-(1-p)\frac{1}{1+\exp[0.1416(f_1(p)-t_d)+0.1054]} \qquad \text{if } f_1(p) > f_2(p)$$

SO and UE assignments are solved in numerical way. Note that all the above three models are subject to maximal allowed diverted volume constraint.

$$p' \le Q_{p,\max} / Q_1$$

3.8 Comparison of Simulation and Analytical Methods

To test whether the analytical delay estimation model can provide satisfactory results, experiments are conducted to compare use delays calculated from the proposed analytical model with those obtained from simulation model for a 1-mile long work zone with one lane closed on the I-83 SB segment. The work zone duration is fixed to 6 hours while the work zone starting time ranges from 0:00 to 23:00.

In this case study, traffic flows are time-varying during a day. Two scenarios with different traffic congestion levels are analyzed. AADT in Scenario 2 is 1.5 times higher than the baseline AADT in Scenario 1. Hourly Traffic Volumes in the two scenarios are provided in Table 3-4 and Figure 3-15. Truck percentage is 10% and the work zone capacity is 1344 vphpl.

Table 3-4 Hourly Traffic Distribution in Two Scenarios

Hours	% of AADT	AADT =19528 Baseline Volume	AADT=29292 High Volume	Hours	% of AADT	AADT =19528 Baseline Volume	AADT=29292 High Volume
00-01	0.79%	155	233	12-13	6.51%	1271	1907
01-02	0.38%	74	111	13-14	6.39%	1248	1872
02-03	0.38%	74	111	14-15	6.39%	1247	1871
03-04	0.37%	73	110	15-16	7.43%	1451	2177
04-05	0.80%	156	234	16-17	8.31%	1623	2435
05-06	1.85%	362	543	17-18	8.51%	1662	2493
06-07	3.88%	757	1136	18-19	6.75%	1318	1977
07-08	6.27%	1225	1838	19-20	4.44%	867	1301
08-09	6.50%	1270	1905	20-21	3.24%	633	950
09-10	4.93%	962	1443	21-22	2.67%	521	782
10-11	4.92%	960	1440	22-23	1.90%	372	558
11-12	5.20%	1016	1524	23-24	1.18%	231	347

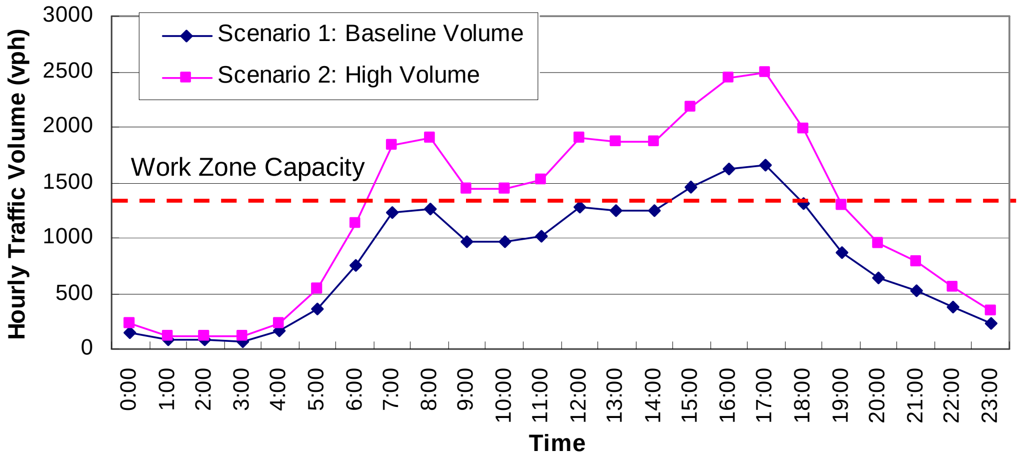


Figure 3-15 Hourly Traffic Distribution in Two Scenarios

Table 3-5 shows the user delays obtained from simulation and the analytical model. The corresponding running times are also listed in this table.

Table 3-5 Estimated User Delay with Different Work Zone Starting Time

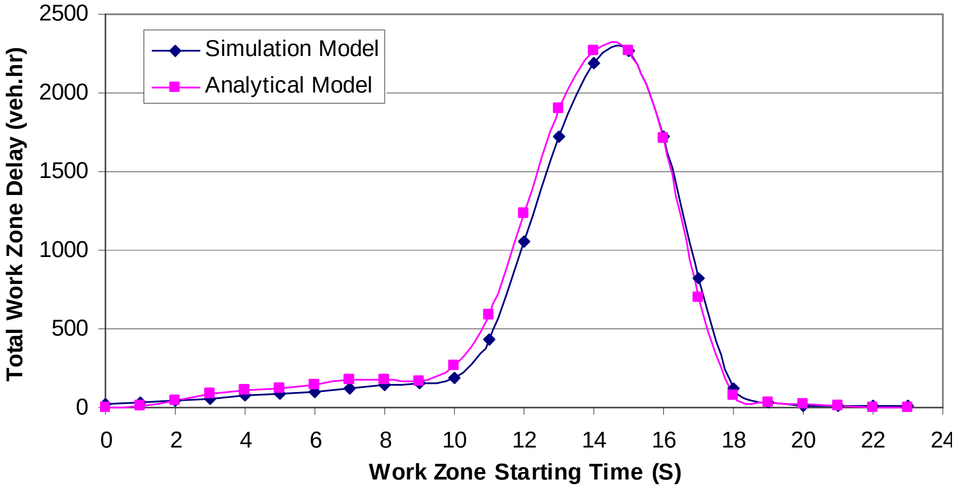
Work Zone Starting Time	Baseline Volume				High Volume			
	Simulation Model		Analytical Model		Simulation Model		Analytical Model	
	Total Delay (veh.hr)	Running Time (sec)	Total Delay (veh.hr)	Running Time (millisec)	Total Delay (veh.hr)	Running Time (sec)	Total Delay (veh.hr)	Running Time (millisec)
0:00	21.77	7.90	4.51	<1	54.74	15.79	9.47	<1
1:00	29.36	8.46	15.11	<1	73.80	19.54	36.69	<1
2:00	44.33	9.58	49.28	<1	297.72	26.81	435.88	<1
3:00	60.07	10.86	86.64	<1	1180.74	47.25	1441.55	<1
4:00	78.33	12.48	106.87	<1	2358.92	73.71	2632.65	<1
5:00	93.70	14.15	126.49	<1	3600.44	100.97	3931.07	<1
6:00	104.16	15.74	146.76	<1	5127.99	135.26	5466.70	<1
7:00	127.36	17.50	173.71	<1	7303.24	189.90	7624.13	<1
8:00	141.77	19.09	175.41	<1	7025.90	182.82	6701.08	<1
9:00	155.73	19.89	171.65	<1	5993.55	176.71	5427.27	<1
10:00	192.63	20.08	267.53	<1	8541.31	255.19	8317.22	<1
11:00	437.15	22.79	586.25	<1	11565.66	398.40	12645.50	<1
12:00	1054.59	29.50	1229.27	<1	12463.96	504.98	16756.40	<1
13:00	1722.91	36.36	1904.79	<1	12562.29	544.54	17404.80	<1
14:00	2188.55	40.92	2266.15	<1	12629.57	568.18	17583.10	<1
15:00	2270.32	40.88	2262.28	<1	13008.43	530.66	17011.00	<1
16:00	1721.66	33.53	1713.74	<1	13094.51	402.36	13440.60	<1
17:00	817.53	21.71	703.26	<1	8490.45	221.10	7194.98	<1
18:00	121.58	12.53	80.04	<1	2285.93	66.87	1551.67	<1
19:00	29.82	9.57	36.31	<1	193.62	21.18	86.82	<1
20:00	12.86	7.51	19.37	<1	32.50	14.02	44.76	<1
21:00	8.48	6.77	10.87	<1	17.08	11.46	24.62	<1
22:00	7.08	6.59	5.37	<1	16.46	11.46	11.55	<1
23:00	14.31	7.36	3.18	<1	32.83	13.82	6.39	<1

The user delay results for Scenario 1 are compared in Figure 3-16 (a). It can be seen that user delays increase sharply when the lane closure occurs in the afternoon peak hour during which traffic demands exceed work zone capacity. The analytical results are close to simulation results and the trend lines are almost the same.

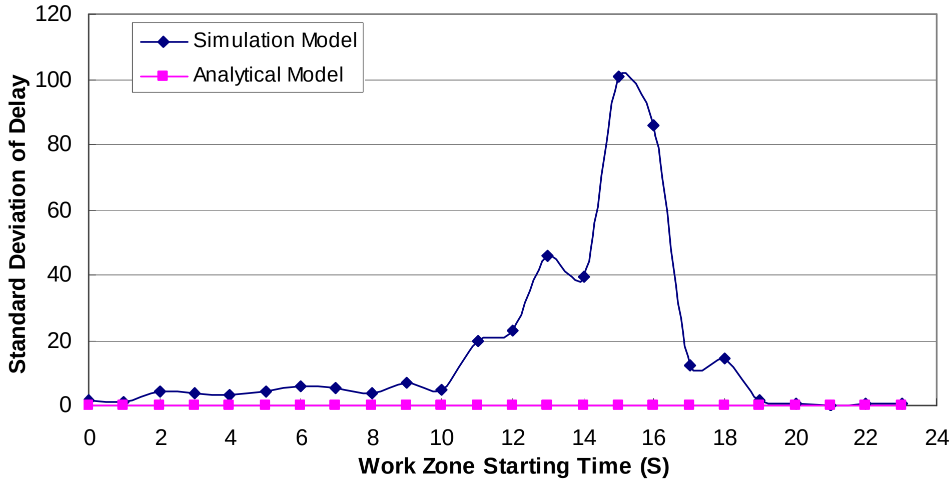
Figure 3-17 (a) displays the obtained user delays for Scenario 2 with higher traffic volumes. The changes of delays become more sensitive to the work zone start time. Placing work zone in morning peak hour or after peak hour would result in severe congestion. The analytical results fit simulation results well accept when work zone starts from 11:00 to 16:00. In these cases delays obtained from CORSIM are fairly insensitive to starting time and the results are much lower than analytical results. After checking the output file and simulation animation of these questionable four cases, we

notice that extremely long queues (more than 11 miles) form and spill back to the entrance node. As discussed in Section 3.5.6, CORSIM is unable to record delays of those vehicles blocked from entering the network at their scheduled time and this limitation leads to the underestimation of user delays in over-saturated conditions. This finding indicates CORSIM results may not be precise for those “bad” work zone schedules that may cause unacceptable queue spillback.

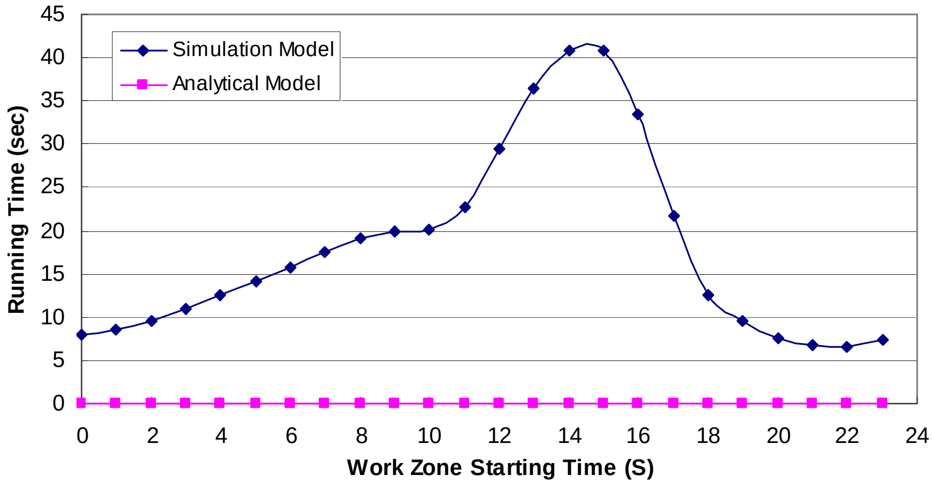
Figure 3-16 (b) and Figure 3-17 (b) show the standard deviation of simulation results with different random seeds in both scenarios. It is obvious that the variability of simulation results increases with the traffic congestion level. The running times needed to obtain the results are compared in Figure 3-16 (c) and Figure 3-17 (c) for Scenarios 1 and 2, respectively. The simulation time is highly related to the network size and traffic congestion level and it ranges from several seconds to hundreds of seconds in this case study. The analytical model obtains a result within 1 millisecond for all cases and the computational time is insensitive to the traffic congestion level. The variation of standard deviation and running time with different work zone schedules indicates that the higher the traffic congestion level, the more simulation replications are needed to obtain a statistically significant result and it takes longer time to complete one simulation replication.



(a) User Delay with Different Work Zone Starting Time (Scenario 1)

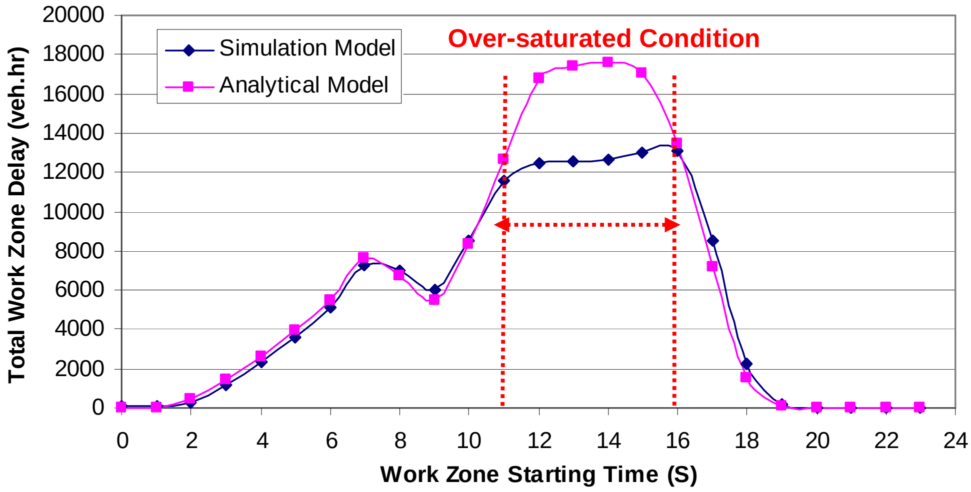


(b) Standard Deviation of User Delays with Different Work Zone Starting Time (Scenario 1)

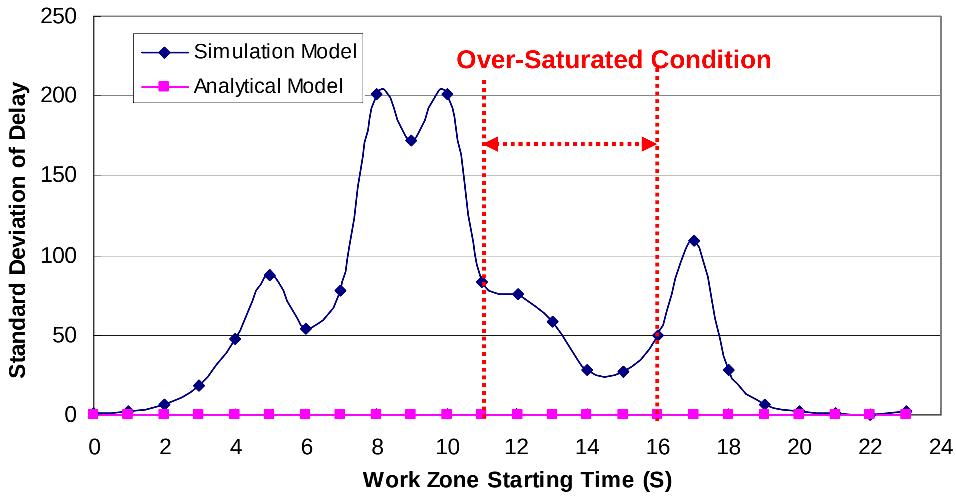


(c) Running Time with Different Work Zone Starting Time (Scenario 1)

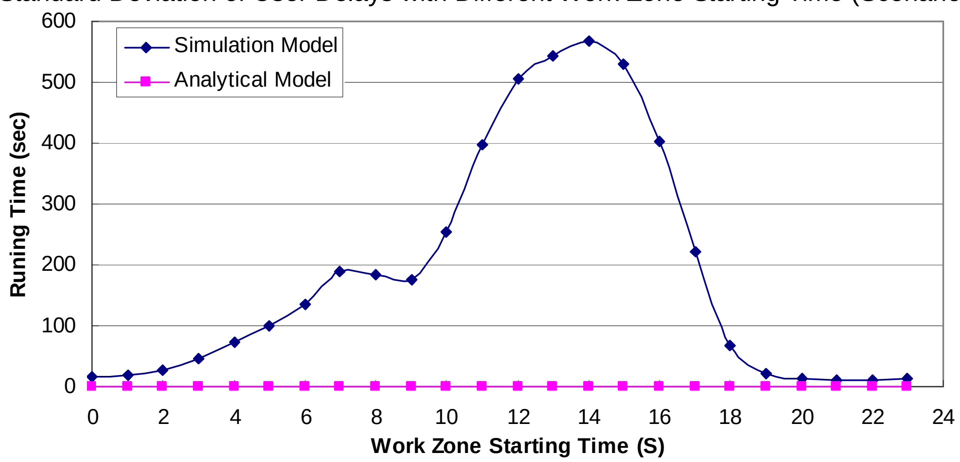
Figure 3-16 User Delay Estimation Results with Varying Work Zone Starting Time (Scenario 1)



(a) User Delay with Different Work Zone Starting Time (Scenario 2)



(b) Standard Deviation of User Delays with Different Work Zone Starting Time (Scenario 2)



(c) Running Time with Different Work Zone Starting Time (Scenario 2)

Figure 3-17 User Delay Estimation Results with Varying Work Zone Starting Time (Scenario 2)

This experiment demonstrates the performance of the proposed analytical model. It is able to quickly estimate work zone delay with a satisfactory precision if important input data, such as work zone capacity and work zone speed, are accurately provided. It has to be noted that the analytical model is established based on a series of simplifying assumptions for a typical simple network in which the links on detour path are aggregated. It may not be applicable in a complex network in which detailed representation of network geometry, traffic characteristics and traffic control plans is required to investigate vehicle interactions with surrounding environment and network-wide impacts.

Chapter 4 Short-term Work Zone Decision Optimization Based on Analytical Model

To search for the optimal or near-optimal combination of critical work zone decisions, two mathematical optimization models are developed for short-duration low-intensity and long-duration high-intensity maintenance projects, separately. The latter model is derived from the former one, taking account of the periodic characteristics of the traffic volumes and the work zone recurrence in real world. The optimization target is to minimize the one-time work zone cost, assuming all decisions to be optimized have no or equal effects on long-term pavement performance of the maintained roadway. A heuristic algorithm is proposed to solve the optimization problem.

4.1 Problem Statement

Work progress in a highway maintenance project can be achieved by providing additional traffic lanes (e.g. strengthening and widening the shoulders along the interstate through the work zone), by designating an alternative route or by providing appropriate lane closures. Although full closure with the use of additional traffic lanes would be the ideal solution with regard to safety, work efficiency and traffic impact, it requires large investment and thus is only limited to major reconstruction projects. When traffic disruption is unavoidable, lane closures have to be carefully designed in order to provide adequate traffic mobility as well as smooth operation of maintenance activities. To aid the decision makers in accomplishing this challenging task, this study develops an optimization model to automatically idenfity the most effective work zone management plan in addressing safety, mobility, constructability, and economy issues.

Using the one-time total work zone cost, including the agency cost and road user cost, to measure the effectiveness of work zone management plans, the optimization model searches for the best plan that minimizes the total work zone cost while satisfying various constraints, such as project deadline and maximum tolerable traffic disruption. In this study, the decisions considered in a candidate work zone management plan include:

(1) How should the road section be divided into work zones? How long and wide should each work zone be?

(2) At what times should the lanes in each work zone be closed and reopened to traffic under time-varying traffic inflows?

(3) Does a traffic impact mitigation strategy, such as accelerating of project execution or diverting traffic to alternative routes, deserve its additional cost?

The above work zone decisions are selected to be jointly optimized for several reasons:

(1) As discussed in Chapter 3, work zone configuration, lane closure schedule, and traffic impact mitigation strategies can significantly influence the one-time total work zone cost. Consequently, good decisions can notably reduce the total work zone cost;

(2) Accelerating work and detour strategy are two of the most frequently used traffic impact mitigation strategies. The former focuses on construction side while the latter is a typical traffic management strategy that can considerably change the demand pattern;

(3) These decisions are highly correlated;

(4) The planner have control over these decisions;

(5) It is difficult to manually find the optimal answers due to large solution space.

To model the above defined optimization problem, the following assumptions have been made:

(1) The lane closure type of each work zone is selected from a set of pre-specified alternatives (e.g. partial closure, full closure or crossover). Each of the lane closure alternative provides a unique combination of three parameters:

- the number of maintained lanes ($N_w^{k_1}$),
- the number of access lanes ($N_a^{k_1}$), and
- the number of usable lanes in opposite direction for crossover operation ($N_c^{k_1}$).
- a binary variable indicating whether detour strategy is employed (b_p)

Note that in practice the lanes to be maintained are usually identical along the longitudinal direction and thus it is assumed that the number of maintained lanes (N_w) is uniform for each work zone.

(2) The time and cost required to complete unit maintenance work can be affected by type of work (e.g. asphalt overlay or full depth replacement), pavement material (e.g. concrete or hot-mix asphalt), pavement thickness (e.g. 8, 10, or 12 inches), construction methods (e.g. sequential method or concurrent method), and resource combinations (e.g. labor or equipment). It is assumed that all decisions influencing pavement service life, such as the type of work, pavement material and thickness,

are given. All the decision variables considered in this model have no impacts or equal impacts on long-term pavement performance.

(3) It may be worth spending more on supplying additional incremental resources of manpower and equipment in order to accelerate the maintenance work and hence reduce the traffic impacts. Usually faster work requires higher cost. Therefore, time-cost tradeoff, represented by a set of work rate options shown in Figure 4-1 is considered in work zone optimization. Each work rate option gives a unique pair of parameters:

- the adjustment (%) of cost required to complete unit length work ($\delta_{z2}{}^{k_2}$);
- the adjustment (%) of time required to complete unit length work ($\delta_{z4}{}^{k_2}$).

The actual production rate when implementing option k_1 can be obtained by the following expression:

$$z_2^{'} = z_2(1+\delta_{z_2}^{k_1}) \qquad \text{Eq.4-1}$$

$$z_4^{'} = z_4(1+\delta_{z_4}^{k_1}) \qquad \text{Eq.4-2}$$

(4) Assuming that maintenance work is performed continuously and all lanes are kept open when no work takes place, work zone length (L) can be derived from work duration (D) given lane closure type and work rate, as shown in Eq.4-3. The relationship between work zone length and duration is shown in Figure 4-2.

$$L = L_f + L_w = L_f + \frac{D - z_3}{z_4 \cdot (1 + f_4 \cdot N_a) \cdot N_w} \qquad \text{Eq.4-3}$$

where, L = the work zone length;
L_f = the fixed setup length;
L_w = the variable work space length;
D = the work zone duration;
z_3 = the fixed setup time;
z_4 = the unit work time per lane-length;
N_w = the number of maintained lanes;
N_a = the number of access lanes (N_A=N-N_W);
f_4 = the multi-lane operation efficiency factor;

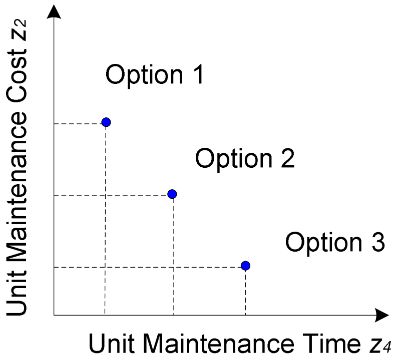


Figure 4-1 Work Rate Options

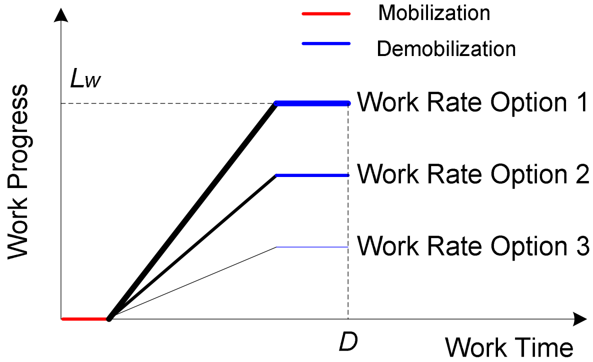


Figure 4-2 Work Progress with Different Work Rate

(5) Work zones are sequential over time and maintenance work is undertaken only on one work zone at a time (Figure 4-3).

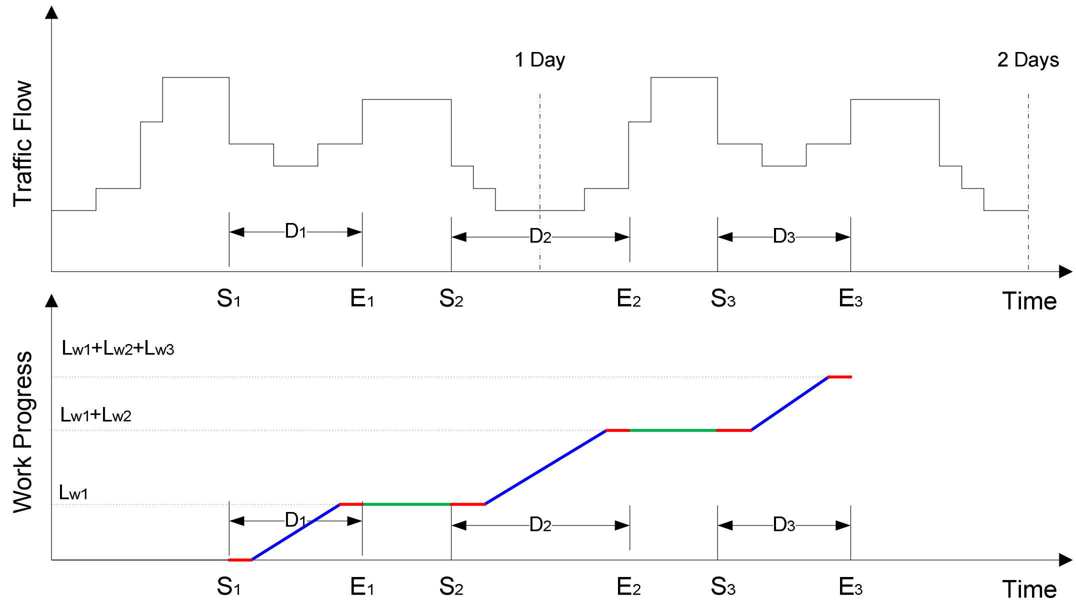


Figure 4-3 Work Zone Schedule

The optimization model is developed for work zones on a multiple-lane two-way highway with a detour, as shown in

(6) Figure 4-4. The road capacities of the maintained road with and without work zone, the detour capacity, and the traffic demand along mainline and detour under normal condition are given. Under normal condition without work zones, the entry inflows do not exceed the roadway capacity. Traffic demand varies over time. An hour is used as a duration unit in which traffic inflows stay appropriately constant.

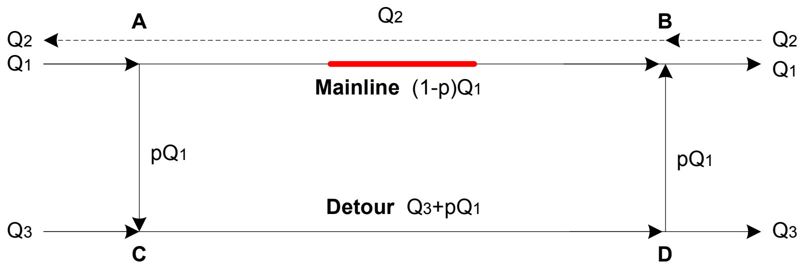


Figure 4-4 Study Network

(7) Traffic management strategies, such as detour strategy and capacity improvement tactic, are considered in the optimization model. A candidate traffic mangement strategy is defined by the following four parameters:

- The fixed employment cost per zone $\beta_1{}^{k_3}$,
- The average additional cost required per unit time $\beta_2{}^{k_3}$;
- Demand management type (u^{k_3}): fixed detour fraction ($u=0$), system control ($u=1$), use choice ($u=2$), or user equilibrium ($u=3$);
- Adjustment of traffic diversion percentage $\delta_p{}^{k_3}$ (%) if the demand management type is "fixed detour fraction";
- Adjustment of the work zone capacity δ_w (%);
- Adjustment of the detour capacity δ_d (%);

(8) The user's time value is represented by a constant average cost per vehicle hour, which is the weighted average cost of driver and passenger user time for different vehicles (passenger cars and heavy vehicles).

4.2 Decision Variables

On a multiple-lane two-way highway, a pavement surface of L_T lane-mile is planed for maintenance within a given time period D. Y types of work zone operation and traffic impact mitigation strategies are considered for employment on this project. Each type of strategy has K_y options ($y=1,\ldots,Y$). For example, there might be K_1 lane closure type alternative, K_2 work rate options, and K_3 detour strategies available for selection. Therefore, the decision variables include:

- The number of work zones m;

- The starting time and ending time of each work zone S_i and E_i (i=1,2,…m);
- The index of the selected option of work zone operation of traffic management alternatives (e.g. lane closure type, work rate option, and detour strategy employed in each work zone) $\vec{K}_i = [k_{1,i}, k_{2,i}, ..., k_{Y,i}]$ (i=1,2,…m, $k_1 \in \{1,2,\ldots, K_1\}$, $k_2 \in \{1,2,\ldots,K_2\}, \ldots, k_Y \in \{1, 2,\ldots,K_Y\}$).

For the sake of presentation, the decision variables representing work zone characteristics are grouped into the following vector:

$$\vec{X} = \begin{bmatrix} \vec{X}_1 \\ \vec{X}_2 \\ \vdots \\ \vec{X}_m \end{bmatrix} = \begin{bmatrix} S_1 & E_1 & k_{1,1} & k_{2,1} & \ldots k_{Y,1} \\ S_2 & E_2 & k_{1,2} & k_{2,2} & \ldots k_{Y,2} \\ \vdots & \vdots & \vdots & \vdots & \vdots \quad \vdots \\ S_m & E_m & k_{1,m} & k_{2,m} & \ldots k_{Y,m} \end{bmatrix}$$

4.3 Objective Functions

4.3.1 General Objective Functions

The optimization objective is to minimize the total cost C_T for the maintenance project. The total cost is the sum of the direct monetary agency cost and indirect user cost. In our proposed model, four types of constraints are considered in optimizing work zone decisions: (1) the total amount of work, (2) the total duration of the project, and (3) the maximal allowable queue length in work zone area. The mathematical optimization model is formulated as follows:

Model 1-1

Objective:

$$\text{Min} \quad C_T(m,\vec{X}) = \sum_{i=1}^{m}[C_{A,i}(\vec{X}_i) + C_{U,i}(\vec{X}_i)]$$

$$= \sum_{i=1}^{m}(C_{M,i}(\vec{X}_i) + C_{S,i}(\vec{X}_i) + C_{I,i}(\vec{X}_i) + C_{D,i}(\vec{X}_i) + C_{V,i}(\vec{X}_i) + C_{E,i}(\vec{X}_i))$$

$$= \sum_{i=1}^{m}[z_1 + z_2^{k_2,i}(1 + f_2 \cdot N_a^{k_1,i}) \frac{(E_i - S_i) - z_3}{z_4(1+\delta_{z_2}^{k_1,i})(1 + f_4 \cdot N_a^{k_1,i})}]$$

$$+ \sum_{i=1}^{m}[\beta_1^{k_3,i} + \beta_2^{k_3,i} \cdot (E_i - S_i)]$$

$$+ v_I \cdot \sum_{i=1}^{m-1}(S_{i+1} - E_i)$$

$$+ v_D \cdot \sum_{i=1}^{m}\int_{S_i}^{E_i}(D^m(t) + D^d(t) + D^p(t))dt$$

$$+ \sum_{i=1}^{m}\int_{S_i}^{E_i}\{v_s[(1-p'(t))Q_1(t) + b_i^c Q_2(t)] + v_d \Delta L_d\, p'(t)Q_1(t) + v_q D_q^m(t)\}dt$$

$$+ \sum_{i=1}^{m}\int_{S_i}^{E_i}\gamma_E v_E D_i^m(t)dt$$

Subject to:

$E_{i-1} \leqslant S_i \leqslant E_i$ (1)

$(E_m - S_1) \leqslant D_T$ (2)

$(E_i - S_i) > z_3 \quad z_2^{'} = z_2(1+\delta_{z_2}^{k_1})$ (3)

$\sum_{i=1}^{m} L_{wi} N_{wi} = L_T$ (4)

$\max\{q(t)\} \le q_{\max}$ (5)

where,

$C_{A,i}$ = Agency Cost of the i^{th} work zone;
$C_{U,i}$ = User Cost of the i^{th} work zone;
$C_{M,i}$ = Agency Maintenance Cost of the i^{th} work zone;
$C_{S,i}$ = Agency Traffic Mitigation Cost of the i^{th} work zone;
$C_{I,i}$ = Agency Equipment/Labor Idling Cost of the i^{th} work zone;
$C_{D,i}$ = User Delay Cost of the i^{th} work zone;
$C_{V,i}$ = User Vehicle Operating Cost of the i^{th} work zone;
$C_{E,i}$ = User Expected Accident Cost of the i^{th} work zone;
z_1 = The fixed setup cost per work zone;
z_3 = The fixed setup time per work zone;
$z_2^{k_{2,i}}$ = The unit length maintenance cost with work rate option k_2 used in zone i;
$z_4^{k_{2,i}}$ = The unit length maintenance time with work rate option k_2 used in zone i;
f_2 = The multi-lane operation cost saving factor;

f_4 = The multi-lane operation time saving factor;
$N_a^{k_3,i}$ = The number of access lanes with lane closure option k_1 used in zone i;
$\beta_1^{k_3,i}$ = The fixed employment cost of detour option k_3 used in zone i;
$\beta_2^{k_3,i}$ = The unit-time employment cost of detour option k_3 used in zone i;
D_i = The duration of zone i;
v_I = The average cost of idling crews and equipments;
v_D = The average value of time per vehicle;
v_s = The average VOC per speed change cycle;
v_d = The average VOC per unit distance;
v_q = The unit queue idling VOC per vehicle;
v_E = The average cost per crash;
γ_E = The estimated number of crashes per 100 million vehicle hours of travel;
b_i^c = The binary variable representing whether crossover operated in zone i;
ΔL_d = The difference between the travel distance on the mainline and on the detour;
$D^m(t)$ = The delay of the traffic on the mainline at the time t;
$D^d(t)$ = The delay of the original traffic on the detour at the time t;
$D^p(t)$ = The delay of the traffic diverted from mainline to the detour at the time t;
$D_q^m(t)$ = The queuing delay of the traffic on the mainline at the time t;
$Q_1(t)$ =Time-varying traffic flow volume in mainline direction 1;
$Q_2(t)$ =Time-varying traffic flow volume in mainline direction 2;
$p'(t)$ =Time-varying traffic diversion rate;
$q(t)$ =Time-varying queue length.

The detailed formulations of the cost components and delay estimations can be found in Chapter 3, and are not duplicated in this chapter.

It can be seen that the total number of variables required to represent a feasible solution is ($6m+1$), including one integer value type decision variable, $2m$ real value type and $4m$ option-type decision variables. The maximum number of work zones m_{max} can be derived from Eq.4-4. Obviously, the longer the project may last, the larger the solution space. Therefore, this general optimization model is more suitable for short-term maintenance projects which can be completed in several days.

$$m_{\max} = \left\lfloor \frac{D_T - L_T \cdot z_{4,\min}}{z_3} \right\rfloor \qquad \text{Eq.4-4}$$

The formulation of the objective function indicates that the agencies costs increase linearly with the number of work zones and the total idling time while the user costs

depend highly on the timing and the lane closure type of each work zone since the former reflects traffic demand level and the latter determines the remaining roadway capacity. Idling time would be beneficial if the additional idling cost can be compensated by the savings on user costs to be obtained by avoiding high traffic periods and closing lanes during low traffic hours.

4.3.2 Modified Objective Functions for Recurrent Work Zones

In practice major maintenance activities such as 4R (Resurfacing, Restoration, Rehabilitation and Reconstruction) projects are usually performed in repetitive time windows considering the periodic characteristic of traffic flows and lane closure time restriction. From a transportation agency's point of view, repetitive operations may reduce work zone traffic impact because of increased driver familiarity and adaptability to the lane drop conditions. From a contractor's point of view, worker may speed-up the operations through learning from recurring practice. Therefore, for major freeway repair, rehabilitation, and reconstruction, work zone characteristics and associated traffic management strategies are often recurrent from day to day or from week to week as demonstrated in Figure 4-5. In these cases it is sufficient to analyze a cyclic period D_T' (e.g. a typical day or week) of work for their traffic impact.

To accommodate implementation requirement of recurring work zone operations, the total work zone cost is calculated as the product of the work zone cost per cyclic period and the total number of periods needed to accomplish the maintenance work:

Model 1-2

Objective:

Min $\quad C_T(m, \vec{X}) = C'_T(m', \vec{X})\dfrac{L_T}{L'_T}$

Subject to:

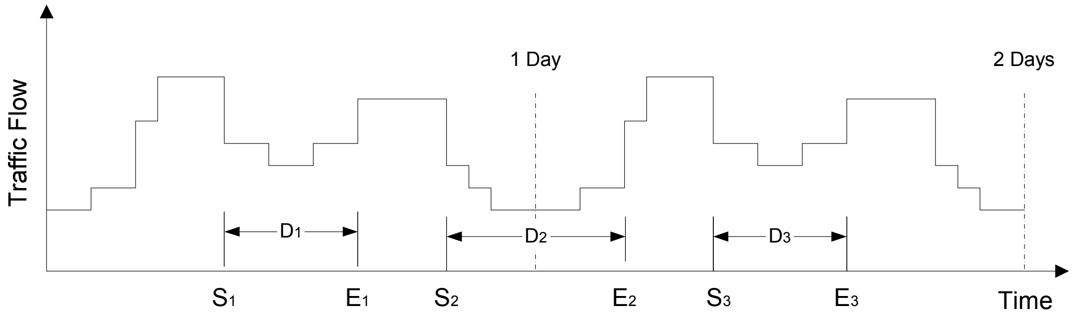
$T_s \leqslant E_{i-1} \leqslant S_i \leqslant E_i \leqslant T_e$ (1)

$(E_{m'} - S_1) \leqslant D_T'$ (2)

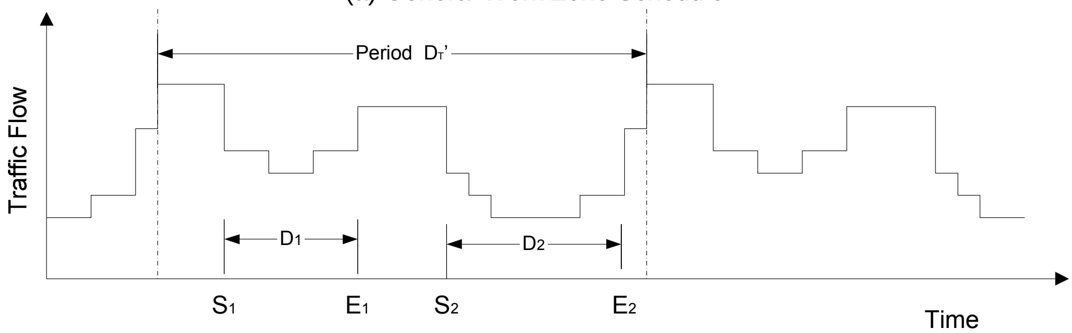
$(E_i - S_i) > z_3$ (3)

$L_T \dfrac{D'_T}{D_T} \leq L'_T \leq L_T \qquad$ where $L'_T = \sum_{i=1}^{m'} L_{wi} N_{wi}$ (4)

$\max\{q(t)\} \leq q_{\max}$ (5)



(a) General Work Zone Schedule



(b) Recurring Work Zone Schedule

Figure 4-5 General and Recurring Work Zone Schedules

With the modified objective function, the number of decision variables is reduced from ($6m+1$) to ($6m'+1$) while the maximum number of m' is reduced from m_{max} to m'_{max}.

$$m'_{\max} = \frac{D'_T}{D_T} m_{\max} \qquad \text{Eq.4-5}$$

It has to be noted that this modified optimization model for recurrent work zones is valid only when the following conditions are satisfied:

(1) Maintenance activities don't extend across two periods. Work zones must be removed before or at the ending time of each period.

(2) The resulting traffic impact cannot be carried from one period to another. That is to say, queues, if any, should be cleared at the ending time of a period.

(3) The idle time between the last work zone in a period and the first work zone in the next period has to be taken into account.

(4) The planning time horizon or the objective lane-length of the project should be long enough to consist of multiple periods so that the difference between two objective functions can be minimized.

4.4 Optimization under Simplified Conditions

With both discrete and continuous decision variables, this work zone optimization problem has a combinatorial nature. If all the discrete decisions, including lane closure type, diversion rate, and production rate are given and the traffic flows are assumed steady over time, the decision variables are reduced to the number of work zones and the duration of each work zone since in this case the traffic pattern doesn't depend on the starting or ending time. When the total lane-mile to be accomplished in the project is sufficient, the integer constraint on the number of wok zones (m) can be released and the optimization mode2 can be simplified to the following problem:

"Find the best length of single work zone so that the cost of unit lane-mile is minimized."

<u>Model 1-3</u>

Objective:

Min $c_t(L) = \dfrac{C_T}{L}$

The total cost of a work zone in which *L* lane-miles of maintenance work are completed can be expressed as:

$$C_T = P_1 + P_2 L + P_3 (\frac{L}{N_w} + L_f)(z_3 + z_4 L) + P_4 (z_3 + z_4 L) + P_5 (z_3 + z_4 L)^2 \quad \text{Eq.4-6}$$

where,

$$P_1 = z_1 + \beta_1 + \beta_2 z_3 \quad \text{Eq.4-7}$$

$$P_2 = z_2 + \beta_2 z_4 \quad \text{Eq.4-8}$$

$$P_3 = (v_D + v_E \gamma_E)[b_1 Q_1 (\frac{1}{V_{w1}} - \frac{1}{V_{f1}}) + b_2 Q_2 (\frac{1}{V_{w2}} - \frac{1}{V_{f2}})] \left\{ (v \frac{1}{V_{w2}} - \frac{1}{V_{f2}})] \right\} \quad \text{Eq.4-9}$$

$$\begin{aligned} P_4 = (v_D + v_E \gamma_E)[b_1 Q_1 (x_1 + x_2 \frac{Q_1}{C_{w1}} + x_3 \frac{Q_1^3}{C_{w1}^3}) + b_2 Q_2 (x_1 + x_2 \frac{Q_2}{C_{w2}} + x_3 \frac{Q_2^3}{C_{w2}^3}) \\ + b_3 Q_3 (\frac{L_{CD}}{V_{CD}^{'}} - \frac{L_{CD}}{V_{CD}}) + b_3 Q_p (\frac{L_{AC}}{V_{AC}} + \frac{L_{CD}}{V_{CD}^{'}} + \frac{L_{DB}}{V_{DB}} - \frac{L_{AB}}{V_{AB}})] \\ + v_s (b_1 Q_1 + b_2 Q_2) + v_d Q_p (L_{AC} + L_{CD} + L_{DB} - L_{AB}) \end{aligned} \quad \text{Eq.4-10}$$

$$\begin{aligned} P_5 = (v_D + v_E \gamma_E + v_q)[b_1 \max(0, \frac{(Q_1 - C_{w1})}{2}(1 + \frac{Q_1 - C_{w1}}{C_{f1} - Q_1})) \\ + b_2 \max(0, \frac{(Q_2 - C_{w2})}{2}(1 + \frac{Q_2 - C_{w2}}{C_{f2} - Q_2}))] \end{aligned} \quad \text{Eq.4-11}$$

$$b_1 = \begin{cases} 1 & if \quad Q_1 > 0 \\ 0 & else \quad Q_1 = 0 \end{cases} \quad \text{Eq.4-12}$$

$$b_2 = \begin{cases} 1 & if \quad N_c > 0 \\ 0 & else \quad N_c = 0 \end{cases} \quad \text{Eq.4-13}$$

$$b_3 = \begin{cases} 1 & if \quad Q_p > 0 \\ 0 & else \quad Q_p = 0 \end{cases} \quad \text{Eq.4-14}$$

Here P_1, P_2, P_3, P_4, and P_5 represent the contributions of the fixed agency cost, unit agency cost, moving delay cost, systematic delay cost along with detour delay cost, and queuing delay cost, respectively.

Note that the demands (Q_1, Q_2, Q_3, Q_p), capacities (C_{w1}, C_{w2}, C_{f1}, C_{f2}, C_{CD}), and time-cost parameters (z_1, z_2, z_3, z_4) used in the above expressions have been updated according to the given diversion fraction, lane closure type and production rate. The traveling speed through work zone (V_{w1},V_{w2}) and the average speed on the detour (V'_{CD}) are calculated with the speed-flow model developed in Chapter 3.

$$V_{w1} = \begin{cases} V_{wf} - \dfrac{Q_1}{C_{w1}}(V_{wf} - V_{wq}) & if \quad Q_1 < C_{w1} \\ V_{wq} & else \end{cases} \qquad \text{Eq.4-15}$$

$$V_{w2} = \begin{cases} V_{wf} - \dfrac{Q_2}{C_{w2}}(V_{wf} - V_{wq}) & if \quad Q_2 < C_{w2} \\ V_{wq} & else \end{cases} \qquad \text{Eq.4-16}$$

$$V'_{CD} = \frac{V_{CD}}{1 + a\left[\dfrac{Q_p + Q_3}{C_{CD}}\right]^b} \qquad \text{Eq.4-17}$$

By dividing the work zone cost (C_T) by the lane-mile (L), the unit cost per lane-mile can be expressed in Eq.4-18. The global optimum of this simplified unconstrained problem can be found at a stationary point, where the first derivative of the objective function is zero. From this analytical method, the optimal work zone length and the corresponding duration can be obtained by Eq.4-20 and Eq.4-21.

$$c_t(L) = \frac{C_T}{L} = \frac{P'_1 + P'_2 L + P'_3 L^2}{L} = \frac{P'_1}{L} + P'_2 + P'_3 L \qquad \text{Eq.4-18}$$

$$\frac{dc_t(L)}{dL} = -\frac{P'_1}{L^2} + P'_3 = 0 \qquad \text{Eq.4-19}$$

$$L^* = \sqrt{\frac{P'_1}{P'_3}} = \sqrt{\frac{P_1 + (P_3 L_f + P_4)z_3 + P_5 z_3^2}{P_3 z_4 / N_w + P_5 z_4^2}} \qquad \text{Eq.4-20}$$

$$D^* = z_3 + z_4 L^* \qquad \text{Eq.4-21}$$

where

$$P'_1 = P_1 + (P_3 L_f + P_4)z_3 + P_5 z_3^2 \qquad \text{Eq.4-22}$$

$$P_2^{'} = P_2 + P_3(z_3 / N_w + z_4 L_f) + P_4 z_4 + 2P_5 z_3 z_4 \quad \text{Eq.4-23}$$

$$P_3^{'} = P_3 z_4 / N_w + P_5 z_4^2 \quad \text{Eq.4-24}$$

4.5 Optimization Algorithm

The work zone decision optimization problem defined in this study seeks to find the best zone division plan, the corresponding starting time and ending time of each zone as well as the combination of available traffic impact mitigation alternatives employed in work zone operation. Thus, there are three classical operation research problems embedded in this study, the cutting stock problem, the scheduling problem and the knapsack problem, which are well-known NP-hard combinatorial optimization problems. In addition, the traffic volumes along mainline and detour routes, denoted as $Q_1(t)$, $Q_2(t)$, *and* $Q_3(t)$, are difficult to model as differentiable continuous functions of time since traffic volumes are discrete data varying over time in reality. Although a closed-form expression for the objective function is provided through numerical calculation of the user costs, the complex and combinatorial nature of the mathematical formulation precludes conventional analytical solution algorithms, such as the branch-and-bound method developed for the mixed integer programming problems. Therefore, a heuristic algorithm called two-stage modified population-based simulated annealing (2PBSA) is proposed to solve the optimization problem.

4.5.1 Basic Concept

Simulated annealing (SA) is a stochastic computational technique derived from statistical mechanics for finding near globally optimum solutions to large optimization problems. The SA algorithm exploits the analogy between annealing

solids and solving combinatorial optimization problems. This neighborhood search algorithm attempts to avoid getting trapped in a local extreme by sometimes moving in a locally worse direction with the purpose of sacrificing short-term fitness to gain longer-term fitness. Through rapidly modifying solution structure within the model, SA often finds high quality candidate solutions in doing refined searches inside prominent regions.

However, when solving realistic problems with a large number of parameters and a great complexity, a great deal of work may be required to reach this neighborhood. For example, when the construction work is allowed in a long-term duration such as a whole week, there may exist several local optimal solutions such as 8-hour off-peak daytime windows, 10-hour nighttime windows, 30-hour weekend windows. Suppose 10-hour nighttime windows are the optimal solution. It may take much time for SA to jump out of other local optima and get close to globally optimal solution, especially when the initial solution is far from it.

Genetic algorithms (GA) are a particular class of population-based evolutionary algorithms (EA) that use mechanisms inspired by biological evolution, such as inheritance, mutation, crossover, and selection. Since the application of GA operators may generate a large jump in the solution space, GAs have a "dynamic" advantage with large possibilities of novel search but nevertheless are not well suited for finely tuned local search.

To overcome the limitation of SA and SA while retaining their strengths, a two-stage population-based simulated annealing (2PBSA) is developed to solve the work zone

decision optimization problem where numerous local optima are likely to occur. The basic idea is to introduce population-based search, elitism and crossover operators in GA into SA so that SA can improve its performance by increasing the solution diversity in the search space.

The procedure of the population-based simulated annealing (PBSA) algorithm is summarized in Table 4-1.

Table 4-1 Population-based Simulation Annealing Algorithm

```
X =Initial solution set X0;                           //Initial solutions
T =Initial temperature T0;                            //Initial temperature
While (T>Tf) {                                        //Check stopping criteria
    For(i=1;i≤N;i++){                                 //Process each solution
        X'i =Modified solution of the ith solution Xi;  //Modify solution
        Δ = C(X'i)-C (Xi);                            //Calculate energy change
        If (Δ≤0){                                     //Check if improved
            Xi = X'i ;                                //Accept new solution if improved
        }
        Else{                                         //Otherwise
            Prob=min(1,e^(-Δ/T));                     //Generate acceptance probability
            α = random(0,1);                          //Generate a random variable
            If (α≤ Prob){                             //Check if acceptable
                Xi = X'i ;                            //Accept new solution if acceptable
            }
        }
    }
    X*=X*';                                           //Update elite archive
    T=T-ΔT;                                           //Reduce temperature
}
Return X*;                                            //Return the results
```

STEP 1:

Generate the first-generation population X with the population size of N. Evaluate each solution X_i and then obtain its objective function value $C(X_i)$. Record the best N^* solutions in the first generation into an elite archive X^*, where N^* is the size of the elite archive.

Set the values of start temperature T_0, stop temperature T_f. and step size of the temperature ΔT. Initialize the current temperature T to T_0.

STEP 2:

As long as the stopping criterion ($T \leq T_f$) is not satisfied, perform the sub-steps.

STEP 2.1: Modify of the solution X_i in the current population to obtain modified solution X'_i. A check procedure is used to ensure the feasibility of the modified solution.

STEP 2.2: Calculate the objective function value and the difference between the objective function value of modified solution and the original solution and value for every solution $\Delta = C(X'_i)\text{-}C\,(X_i)$. If, $\Delta < 0$ go to step 2.4. Otherwise, go to step 2.3.

STEP 2.3: ($\Delta > 0$) Accept the modified solution X'_i with the probability $\exp(-\Delta/T)$.

STEP 2.4: ($\Delta < 0$) Accept the modified solution X'_i with the probability 1.

STEP 2.5: If $i < N$, $i = i+1$, go to STEP 2.1. For each candidate solution in the current generation, repeat the above steps.

Else if $i = N$, then update the best N^* best solutions in the elite archive X^* and reduce temperature $T = T\text{-}\Delta T$, and go to STEP 2.1.

STEP3:

Output the best solution(s) ever found. Transmit this solution or this group of solutions to the next stage.

There are two stages in this newly-developed algorithm, as shown in Figure 4-6. The first stage is initial optimization. In this step, a population-based search procedure in combination with the annealing technique is employed to obtain an initially optimized solution after a wide search in the relatively large solution space. This stage will focus on searching for the best zone division and scheduling plan. A given number of best solutions ever found are saved in an elite archive and they are provided to the next stage as initial solutions.

The second stage is refined optimization. In this step, the same population-based search procedure (PBSA) algorithm is applied with a smaller population size and elite archive size than in the first stage. We seek to use this neighborhood search algorithm to find high quality candidate solutions in doing refined searches inside prominent regions provided by the first stage. The second stage puts more efforts on finding the optimal traffic impact mitigation strategies.

The procedure of the population-based simulated annealing (PBSA) algorithm is described in detail in the next section. The PBSA used in two stages differs in the population size, elite archive size, initial solution generation method, and neighborhood solution generation method.

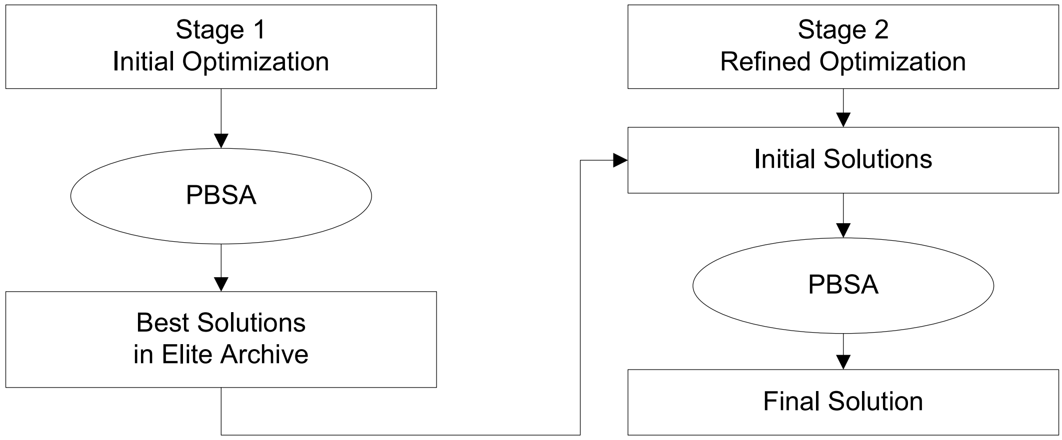


Figure 4-6 Two Stages in the 2PBSA Algorithm

4.5.2 Algorithm Procedure

4.5.2.1 Initial Solution Generation

Initial solutions are generated by imitating engineers' design process. The first step is to identify all off-peak time windows during which the Volume/Capacity ratio is less than a pre-specified value, as shown in Figure 4-7. The work zone schedule of each initial solution is then randomly generated according to the following rules:

(1) Work zones are sequentially scheduled in all off-peak time window within the analysis period [T_s, T_e];

(2) Work zones are scheduled only in off-peak time windows with the lowest average traffic volume;

(3) Work zones are scheduled only in the off-peak time windows with the longest duration;

(4) Work zones are scheduled in randomly selected time windows.

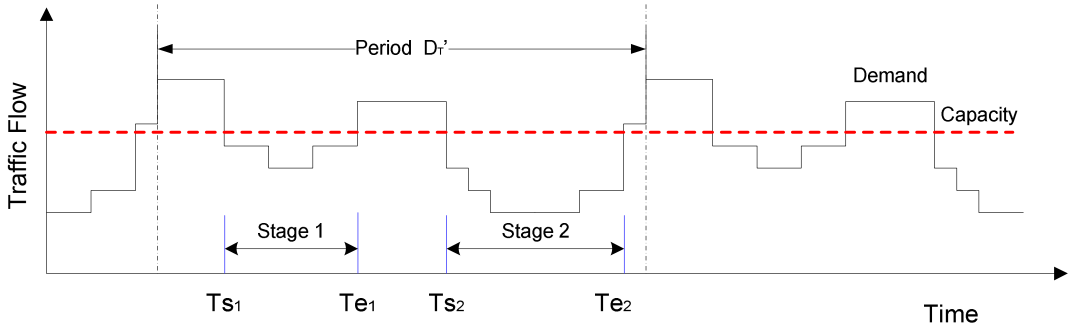


Figure 4-7 Time stage Identification

For each work zone in an initial solution, randomly select the discrete decision variables, which are IDs of each work zone strategy type. Once all continuous and discrete decision variables are determined, necessary characteristics are known for each work zone in an initial solution. In order to improve the quality of an initial solution, the optimization model under simplified condition (Model 1-3) is applied for each work zone in the initial solution using average traffic information during the time stage. Based on the best length and duration obtained from Eq.4-20 and Eq.4-21., each work zone can be extended or divided into multiple sub-zones. After this pre-optimization process, an initial solution is created. The procedure to create initial solutions in the first generation is demonstrated in Figure 4-8.

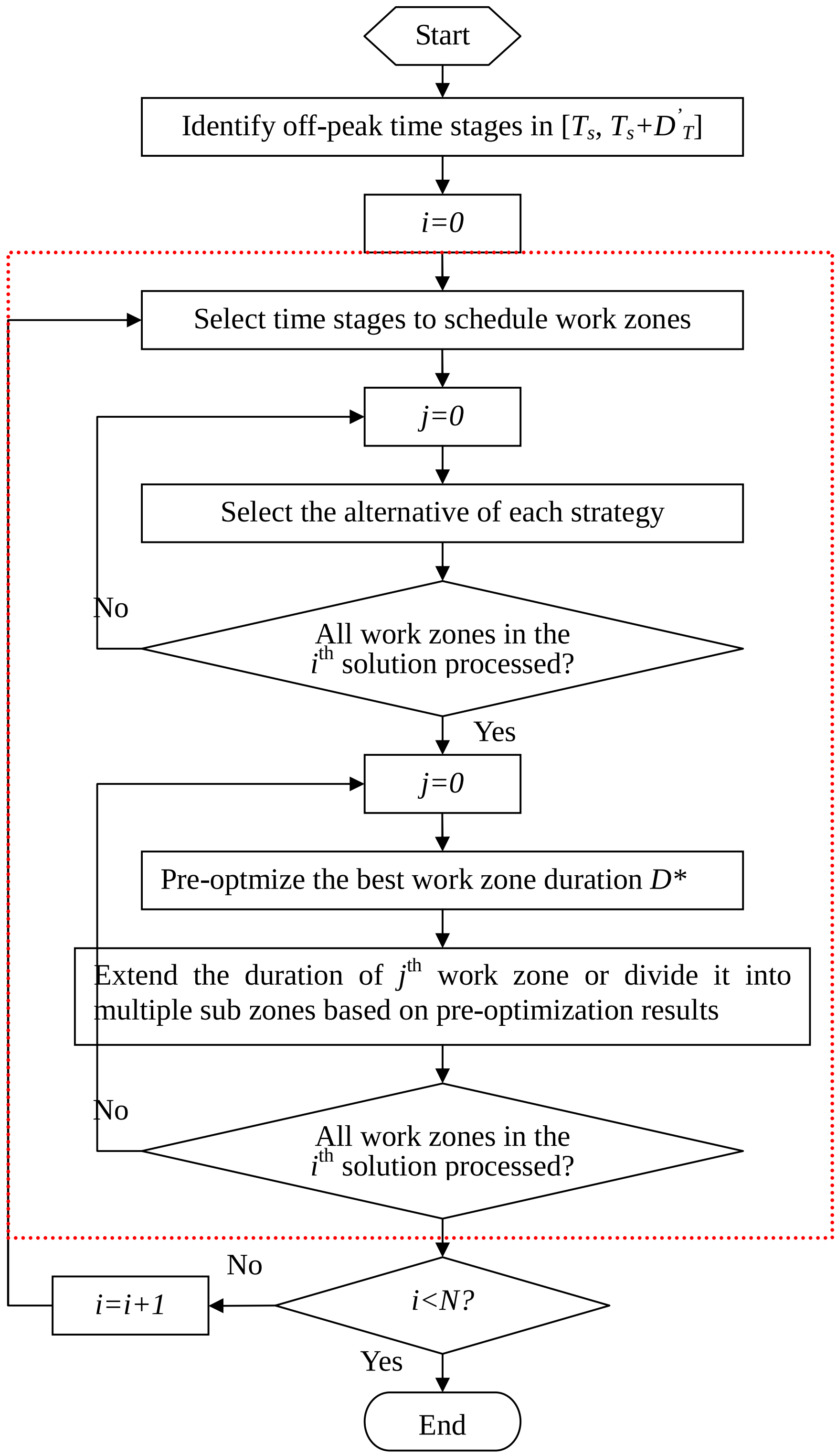


Figure 4-8 Procedures to Create Initial Solutions

4.5.2.2 Constraint Handling Method

The objective function in the modified optimization model 2 is subject to the following five constraints:

(1) Work zone sequence constraint: All work zones are sequential over time;

(2) Cyclic period constraint: all planned work zones are within the same cycle period;

(3) Minimum work zone duration constraint: each work zone has to have enough time for the mobilization/demobilization and work zone setup/removal;

(4) Lane-mile constraint: The total lane-mile completed in one cyclic period should be long enough to ensure that the project can be finished on time. There is no need to complete more lane-miles than the agency requires.

(5) Traffic impact constraint: the resulting queue length should not exceed the maximum acceptable queue length.

The way that the proposed algorithm generates solutions will ensure the satisfaction of the first three constraints. Two methods are used to handle the violation of the last two constraints:

(1) Repairing Method

Before evacuating a solution, the lane-mile constraint is checked. If the lane-mile completed in one period exceeds the objective lane-mile of the project ($L'_T > L_T$), the solution will be fixed by decreasing the length of the first or last work zone. The repair procedure is demonstrated in Figure 4-9.

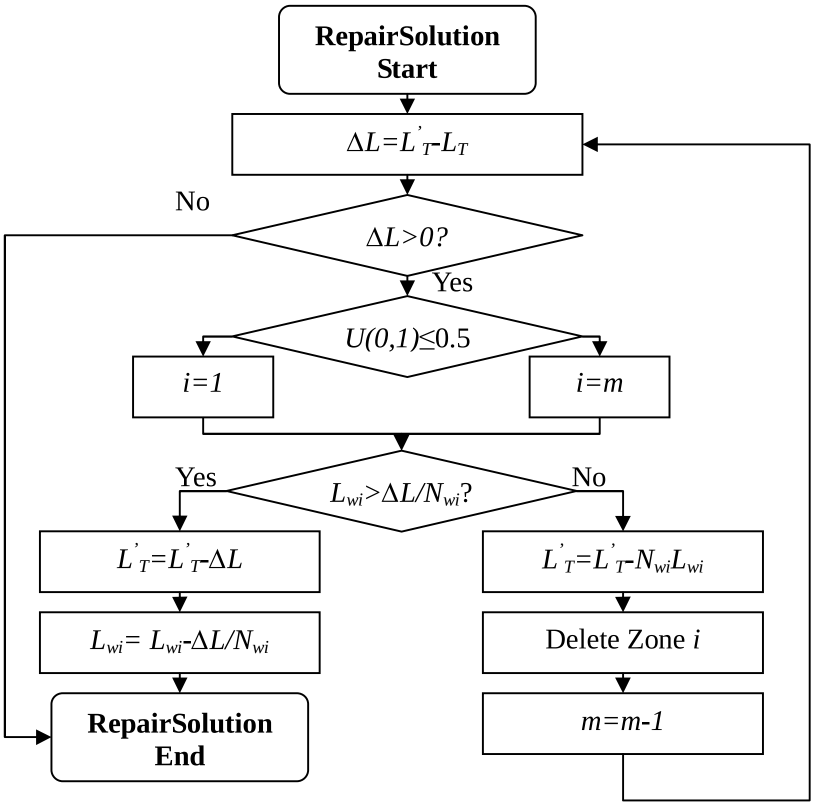


Figure 4-9 Solution Repair Procedure

(2) Penalty Method

If the lane-mile completed in one period is too short to complete the whole project on time ($L'_T < L_T D'_T / D_T$) or the queue length exceeds the allowable limit ($\max(q(t)) > q_{max}$), penalty costs are added to the objective function value based on the severity of constraint violation.

$$C_T(m,\vec{X}) = C'_T(m',\vec{X})\frac{L_T}{L'_T} + C_{P1} + C_{P2} \quad \text{Eq.4-25}$$

$$C_{P_1} = \lambda_1 \max\{0,[\max(q(t)) - q_{\max}]\} \quad \text{Eq.4-26}$$

$$C_{P_2} = \lambda_2 \max\{0,(D'_T\frac{L_T}{L'_T} - D^T)\} \quad \text{Eq.4-27}$$

4.5.2.3 Solution Evaluation and Elite Archive

In each generation, the objective function value, which is the sum of the one-time work zone cost and the penalty costs, is evaluated for each individual solution in the

population. A reduction in the objective function value corresponds to a better solution to this minimization problem. An external archive is used to store a set of best-known solutions, which are updated at each generation.

4.5.2.4 New Solution Generation

At each step, the current population is component of N solutions $\{\vec{X}_1, \vec{X}_2, \cdots, \vec{X}_N\}$ and each solution lists the characteristics of mi work zones as shown in Figure 4-10.

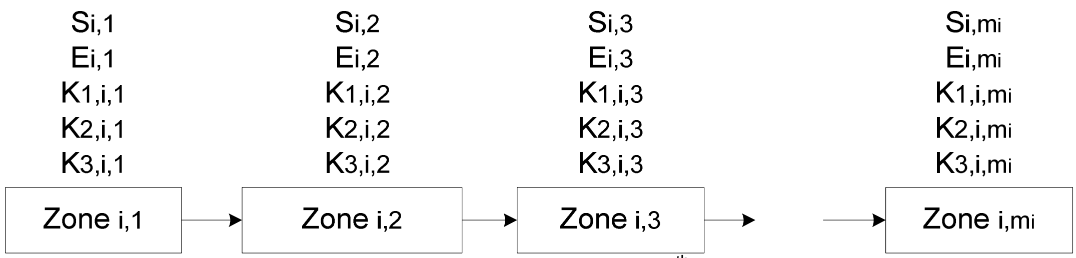


Figure 4-10 Data structure of the i^{th} Solution

New solutions are generated from current solutions through four problem-specific operators, of which three are neighborhood-based mutation operators and one is crossover operator.

(1) Mutation Operator 1

With a pre-specified probability, a neighborhood solution is generated from the old solution through increasing or decreasing the duration of a randomly selected work zone. The increase/decrease event can occur at either the beginning or the ending of the selected work zone. The preceding zone, the selected zone, or the following zone might be deleted or merge with each other to avoid the violation of minimum work

zone duration constraint. The detailed procedures are illustrated from Figure 4-11 to Figure 4-15.

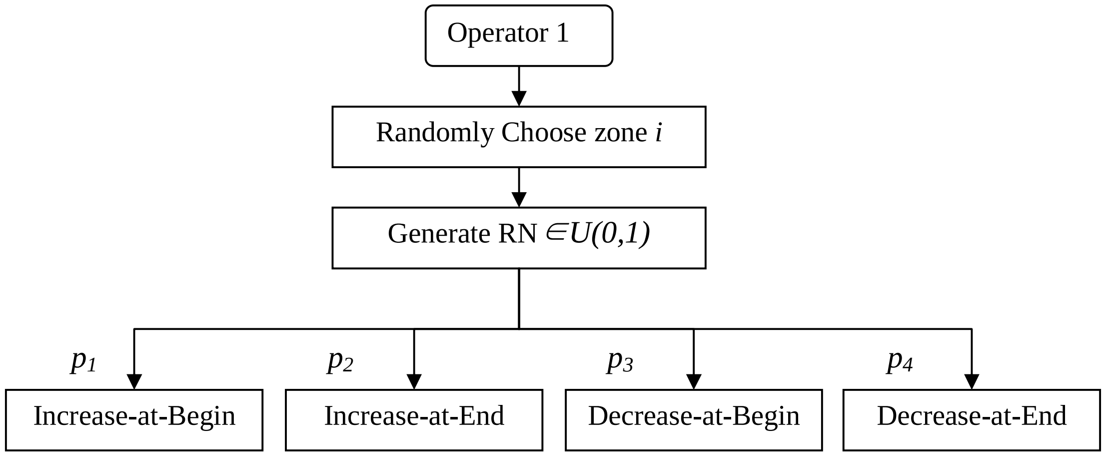


Figure 4-11 Procedure of New Solution Generation Operator 1

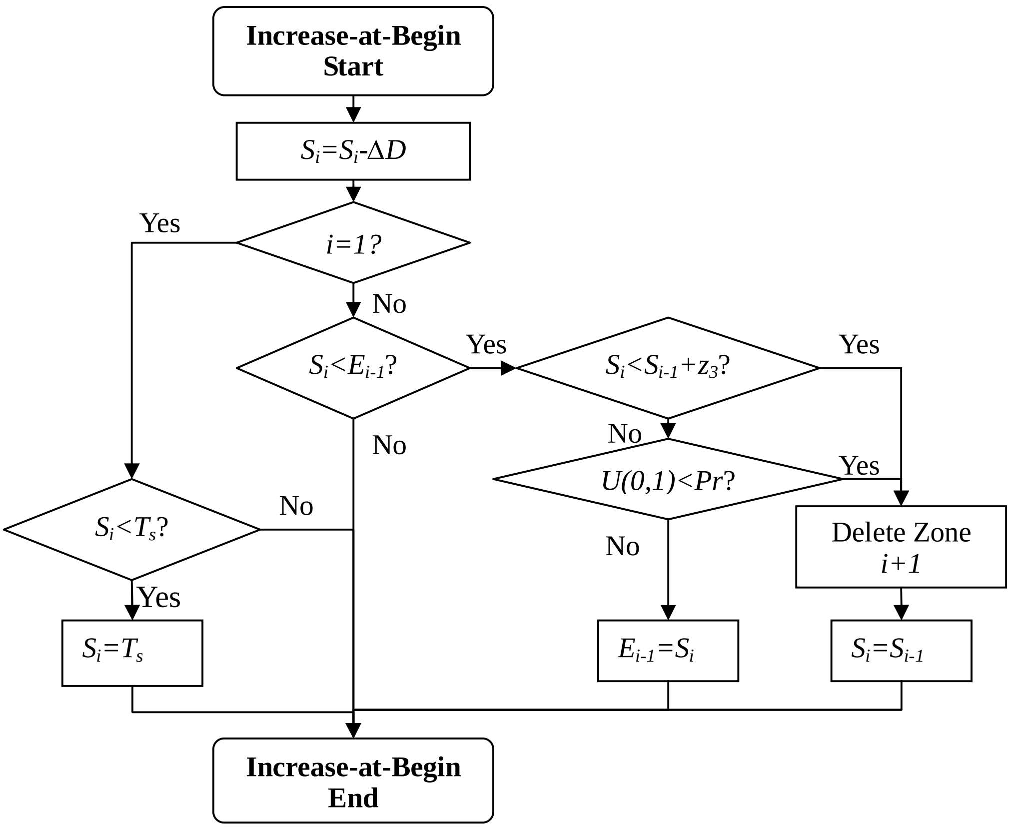


Figure 4-12 Increase-at-Begin Event

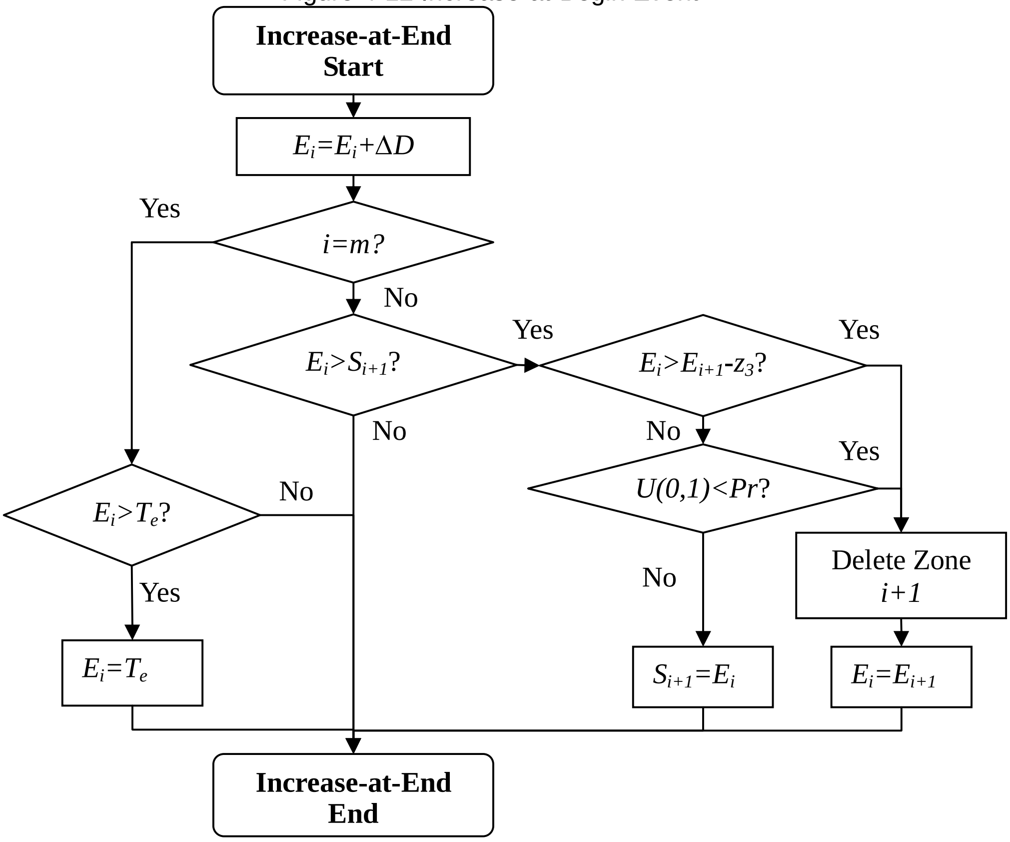
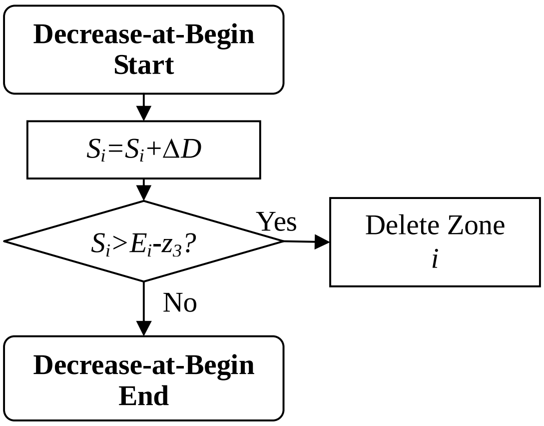


Figure 4-13 Increase-at-End Event



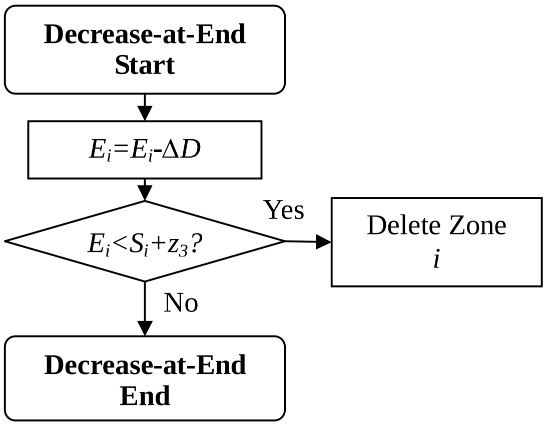


Figure 4-14 Decrease-at-Begin Event

Figure 4-15 Decrease-at-End Event

(2) Mutation Operator 2

With Operator 2, a new solution is obtained from an old one by dividing a work zone into two separate zones or by combing two zones to form a longer construction window.

(3) Mutation Operator 3

With a probability, a new solution is generated from one of the current best solutions ever found through modifying the alternative ID of one or more work zone operation and traffic management strategies in a randomly selected work zone.

(4) Crossover Operator 4

Maintaining population diversity is crucial to the PBSA’s ability to explore different regions of the search space and escape local optima. A one-point crossover operator comes into play to create a new solution through recombining work zone information of the old solution and a randomly selected elite solution in the external archive. A single crossover time point (T_c) is randomly selected between the period starting time (T_s) and period ending time (T_e). All work zone information beyond that point is swapped between the two parent solutions. The resulting offspring is the new solution. The procedure is illustrated in Figure 4-16.

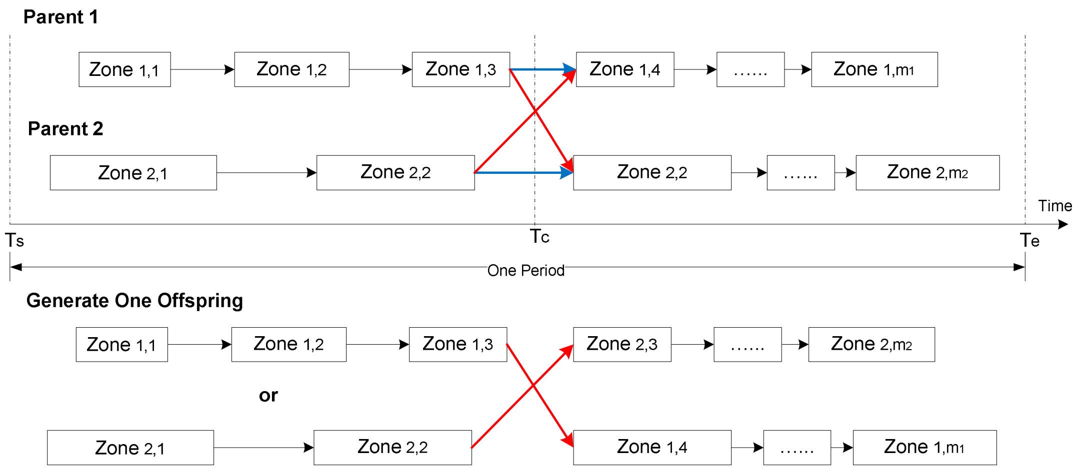


Figure 4-16 One-point Crossover Operator

As shown in Figure 4-17, a work zone in parent solution 1 can be shortened or merged with another work zone in parent solution 2 at crossover time point (T_c). Note that the new work zone will be deleted if minimum work zone duration constraint is violated.

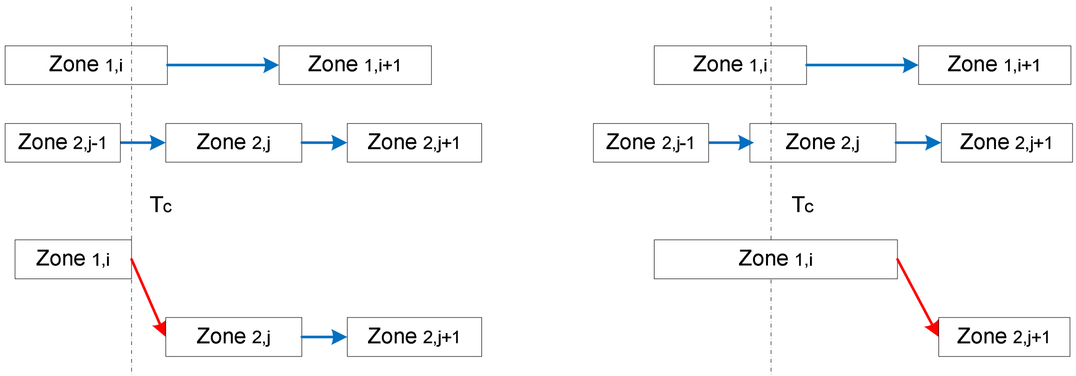


Figure 4-17 Recombination of Work Zones

Note that the probability of applying each mutation or crossover operator is different in two stages of the proposed algorithm. Since the first stage focuses on widely search of the solution space while the second stage seeks to fine-tune local optimums

obtained from the first stage, mutation operations are employed more frequently in the second stage.

4.6 Numerical Examples

In order to demonstrate that the proposed methodology can function effectively in handling work zone optimization problem, a numerical experiment is conducted based on a hypothetical maintenance project on a segment on the United States Route I-95 north bound in Maryland.

4.6.1 Test Network

The study network consists of the eight-lane two-way I-95 corridor northbound, the parallel four-lane two-way arterial US 1, and two highways connecting them, MD 32 eastbound and MD 175 westbound, as shown in Figure 4-18. Link information and weekday traffic distributions on the mainline and detour routes are provided in Table 4-2 and Table 4-3. The baseline values of input parameters are provided in Table 4-4.

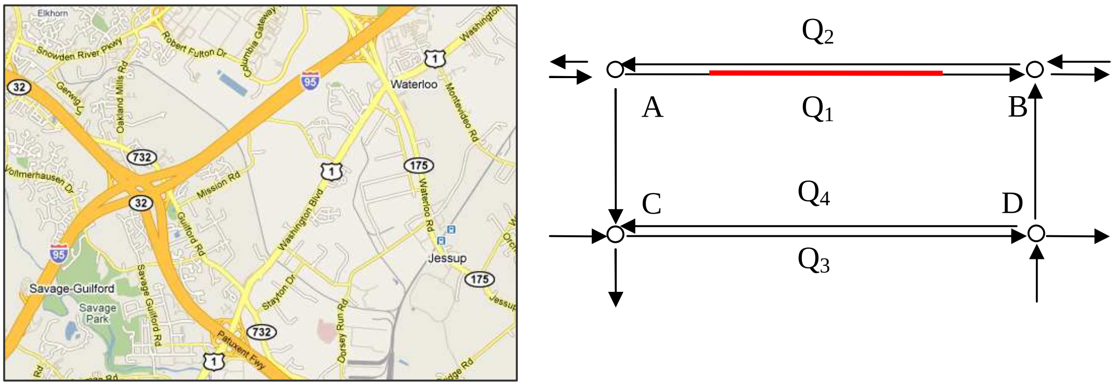


Figure 4-18 Study Network Geometry

Table 4-2 Study Network Link Information

Link	Length(*L*) (mile)	# of Lanes(*N*) (#)	Capacity(C_f) (vphpl)	Free Flow Speed (V_f) (mph)
AB/BA	3.1	4	2,200	65
AC	1.8	2	1,900	55
CD/DC	2.76	2	1,900	40
DB	0.6	3	1,900	55

Table 4-3 AADT and Hourly Traffic Distribution on the Mainline and Detour Routes

Hour	Time	Mainline AB	Mainline BA	Detour CD	Detour DC
		Q1	Q2	Q3	Q4
0	0:00-1:00	1,052	1,220	217	232
1	1:00-2:00	758	772	149	208
2	2:00-3:00	603	694	134	215
3	3:00-4:00	603	754	134	209
4	4:00-5:00	911	1,491	172	325
5	5:00-6:00	2,049	4,237	286	562
6	6:00-7:00	3,806	6,951	517	696
7	7:00-8:00	6,033	7,554	765	1,166
8	8:00-9:00	6,724	6,171	849	1,000
9	9:00-10:00	5,624	4,475	922	872
10	10:00-11:00	4,896	4,978	822	832
11	11:00-12:00	5,001	4,619	796	889
12	12:00-13:00	5,064	4,831	1,115	1,020
13	13:00-14:00	5,108	4,725	1,034	996
14	14:00-15:00	6,119	5,344	1,003	1,003
15	15:00-16:00	7,096	5,764	1,141	1,027
16	16:00-17:00	7,444	6,395	1,302	1,072
17	17:00-18:00	7,197	7,023	1,393	1,215
18	18:00-19:00	7,013	5,735	849	728
19	19:00-20:00	5,285	4,298	556	621
20	20:00-21:00	3,810	3,636	470	522
21	21:00-22:00	2,972	3,264	347	399
22	22:00-23:00	2,377	2,853	282	291
23	23:00-24:00	1,775	1,775	251	242
AADT		99,314	99,552	15,501	16,336
Average Hourly Volume		4138	4148	645	680
Truck Percentage		5%	5%	0%	0%

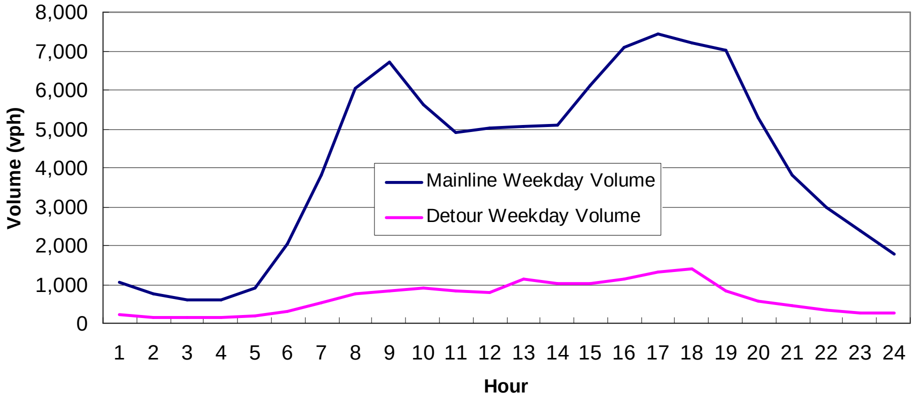


Figure 4-19 Traffic Distributions over the Mainline Route and Detour

Table 4-4 Notation and Baseline Numerical Inputs

Variable	Description	Value
C_w	Work Zone Capacity	1340 vphpl
V_{wf}	Work zone speed limit	55 mph
V_{wq}	Average work zone speed at full capacity	43 mph
L_f	Fixed work zone setup length	0.1 mile
s_d	Average deceleration distance	1 mile
a_a	Average acceleration rate	5.59 mph/s[1]
v_I	Average cost of idling crews and equipments	2000 $/hr
v_{D_car}	Average value of time for passenger cars	16 $/veh·hr[2]
v_{D_trucks}	Average value of time for trucks	27 $/veh·hr[2]
v_{s_car}	Average speed change VOC for passenger cars (65 mph – 55 mph – 65 mph)	0.037 $/veh.cycle[2]
v_{s_truck}	Average speed change VOC for trucks (65 mph – 55 mph – 65 mph)	0.051 $/veh.cycle[2]
v_{q_car}	Average queue idling VOC for passenger cars	1.00 $/veh.hr[2]
v_{q_truck}	Average queue idling VOC for trucks	1.12 $/veh.hr[2]
v_d	Average VOC per unit distance	0.32 $/mile[3]
v_E	Average cost per crash	142,000$/accident[4]
γ_E	Estimated number of crashes per 100 million vehicle hours of travel	40 acc/100mvh[4]
l_{veh}	Average vehicle length	20 feet
$Q_{p,max}$	Maximum allowed diverted volume	1800 vph
QL_{max}	Maximum acceptable queue length	1.5 mile
D_T'	The duration of an cyclic analysis period	24 hours (1 weekday)
T_s	Starting time of an analysis period	16:00 pm
T_e	Ending time of an analysis period	16:00 pm next day
D_T	The maximum acceptable number of periods	50 periods

[1]Source: Shibuya, S., T. Nakatsujji, T. Fujiwara. “Traffic Control at Flagger-Operated Work Zones on Two-Lane Roads.” Transportation Research Record 1529, pp. 3-9, 1996
[2]Source: NCHRP Report 133 “Procedures for Estimating Highway User Costs, Air Pollution, and Noise Effects.”, 1972. All the costs factors have been converted to 2008 values by multiplying older costs rates by escalation factors derived from the Consumer Price Index (CPI).
[3]Source: Rister, B. W., C. Graves. “The Cost of Construction Delays and Traffic Control For Life-Cycle Cost Analysis of Pavements.” KTC-02-07/SPR197-99 & SPR218-00-1F, Kentucky Transportation Center. 2002
[4]Source: Chien, S. and P. Schonfeld. “Optimal Work Zone Lengths for Four-lane Highways,” Journal of Transportation Engineering, Vol. 127, No. 2, pp. 124-131, 2001

4.6.2 Experiment Design

The task of the hypothetical project is to maintain all four lanes within a 2-mile long section of northbound I-95. With two candidate lane closure type (single-lane closure and double-lane closure), three kinds of work zone management strategies are

considered as candidate measures for reducing traffic impacts, including accelerating construction, guiding traffic to use alternative route through ITS equipments and employing advanced merge control system. The alternatives of each candidate strategy are listed in Table 4-5.

When testing the optimization model, we give special attention to the following four scenarios:

Scenario 1: High volume traffic level, high intensity work (no detour control)

Scenario 2: Low volume traffic level, high intensity work (no detour control)

Scenario 3: High volume traffic level, low intensity work (no detour control)

Scenario 4: Low volume traffic level, low intensity work (no detour control)

The baseline traffic volume provided in

Table 4-4 is considered as high volume level while the same traffic distribution with 60% of the baseline AADT is used in low volume level scenarios. Work intensity is measured by the amount of work to be done in unit area. Usually, the higher the work intensity, the more worker and equipments are needed in the work zone and it takes higher cost and longer time to process unit area of roadway. The work intensity highly depends on the type of maintenance work. In this experiment, pothole patching, a routine maintenance activity, is used to test scenarios with low intensity work and asphalt resurfacing, a rehabilitation activity, is used in high intensity work scenarios. The information of traffic levels and work types are listed in

Table 4-6 and

Table 4-7. Since in practice most work zone management plan design does not involve the analysis of controlled user detour behavior, the advanced detour control option is deactivated in the above four test scenarios. The impact of user route choice behavior on the optimized work zone management plan will be discussed in the sensitivity analysis.

Table 4-5 Candidate Work Zone Management Strategies

Strategy Type 1: Lane closure configuration					
Alt #	***Description***	N_W		N_a	N_c
1	Single Lane Closure	1		0 (use shoulder)	0
2	Double Lane Closure	2		0 (use shoulder)	0
Strategy Type 2: Accelerated construction					
Alt #	***Description***	Δz_1	Δz_2	Δz_3	Δz_4
1	Normal work rate	0%	0%	0%	0%
2	Medium work rate	0%	+10%	0%	-15%
3	High work rate	0%	+20%	0%	-30%
Strategy Type 3: Detour strategy					
Alt #	***Description***	***Behavior Model***		β_1 ***($/zone)***	β_2 ***($/hr)***
1	No detour control	-		0*	0*
2	Advanced detour control	SO/RC/UE		500	200
Strategy Type 4: Merge control system					
Alt #	***Description***	***Capacity Change***		β_1	β_2
1	No merge control	0%		0*	0*
2	Advanced merge control	+15%		100	50

*Included in unit agent cost and time, z_2 and z_4.

Table 4-6 Traffic Conditions with Different Congestion Levels

Traffic Level	***Traffic Volume Multiplier***	**AADT**
Low	0.6	59,588
High	1.0	99,314

Table 4-7 Work Types with Different Work Intensity

Work	***Description***	z_1 ***$/zone***	z_2 ***$/lane.mile***	z_3 ***hr/zone***	z_4 ***hr/lane.mile***
Low	Pothole Patching	1000	10,000	2	4
High	Resurfacing	1000	110,000	2	8

Sensitivity analysis is conducted in order to explore how variations in key input parameters affect the optimization result. The major test parameters include traffic

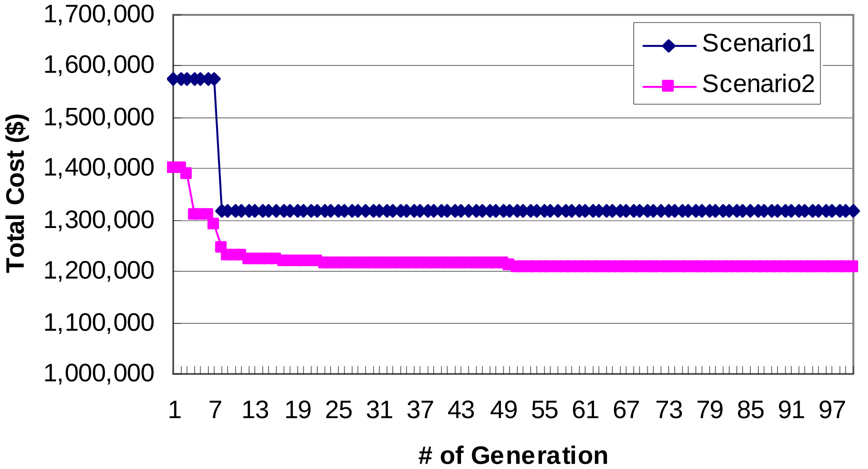
volume, work intensity, idling cost, project deadline, analysis period, and user route choice behavior model.

4.6.3 Optimization Results

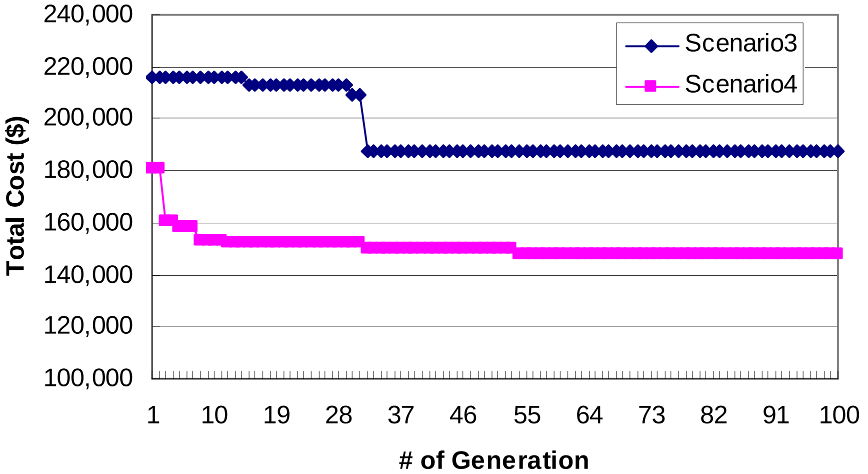
(1) Convergence Analysis

After running 50 generations at the first stage with the population size of 100 and another 50 generations at the second stage with the population size of 20, the optimization process converges to an optimized solution in each test scenario, as shown in Figure 4-20. A monotonically decreasing relation appears since the algorithm records the best solution ever found at each generation. It can be seen that dramatic improvements of objective function value always occur at the early phase of the first stage and those elite solutions obtained from the first stage may be improved through the refined search conducted at the second stage.

Table 4-8 provides the optimized decision variables and the associated cost information for each scenario. Mainly because the project (up to 50 periods) deadline is not tight and idling cost (2000$/hr) is not high enough to sacrifice traffic mobility, the optimized lane closure strategy in all scenarios is to make sure that the reduced roadway capacity is still able to accommodate the traffic most of the time. Therefore, only minor queues appear on the mainline route and the major user cost component is the moving delay cost due to reduced work zone speed. Note that advanced merge control is selected in almost all work zones because improved work zone capacity can extend the work time and consequently save agency cost.



(a) Convergence for Scenarios 1 and 2 with High Intensity Work



(b) Convergence for Scenarios 3 and 4 with Low Intensity Work

Figure 4-20 Optimization Convergence for Four Test Scenarios

Table 4-8 Optimized Solutions in Four Scenarios

(a) Optimization Result of Scenario 1 (High Traffic Level, High Intensity Work)

Zone No.	***Starting Time***	***Ending Time***	***Duration***	***Length***	***Str#1***	***Str#2***	***Str#3***	***Str#4***
(#)	(0-24)	(0-24)	(hr)	(mile)	Lane Closure	Work Rate	Detour	Merge Control
1	20:30	6:30	10	0.63	Double	Normal	No	Yes

C_M($)	1,065,600	*C_D($)*	19,768
C_S($)	5,760	*C_V($)*	1,530
C_I($)	240,800	*C_E($)*	1,421
Agent Cost C_A($)	1,312,160	*User Cost C_U($)*	22,719
Total Cost($)			1,334,879
Lane-mile/Period			1.25
# of needed Periods			9.6(≈10)
Maximal Queue Length (mile)			1.38

(b) Optimization Result of Scenario 2 (Low Traffic Level, High Intensity Work)

Zone No.	***Starting Time***	***Ending Time***	***Duration***	***Length***	***Str#1***	***Str#2***	***Str#3***	***Str#4***
(#)	(0-24)	(0-24)	(hr)	(mile)	Lane Closure	Work Rate	Detour	Merge Control
1	19:30	7:00	11.5	0.74	Double	Normal	No	Yes
2	7:00	11:00	4	0.25	Single	Normal	No	Yes
3	11:00	15:00	4	0.25	Single	Normal	No	Yes

C_M($)	1,140,660	*C_D($)*	23,296
C_S($)	7,710	*C_V($)*	2,215
C_I($)	45,425	*C_E($)*	2,523
Agent Cost C_A($)	1,193,795	*User Cost C_U($)*	28,035
Total Cost($)			1,221,830
Lane-mile/Period			1.98
# of needed Periods			6.04((≈6)
Maximal Queue Length (mile)			1.17

(c) Optimization Result of Scenario 3 (High Traffic Level, Low Intensity Work)

Zone No.	***Starting Time***	***Ending Time***	***Duration***	***Length***	***Str#1***	***Str#2***	***Str#3***	***Str#4***
(#)	(0-24)	(0-24)	(hr)	(mile)	Lane Closure	Work Rate	Detour	Merge Control
1	21:00	6:00	9	1.56	Double	Fast	No	Yes

C_M($)	119,040	*C_D($)*	5846
C_S($)	2,112	*C_V($)*	524
C_I($)	60,000	*C_E($)*	426
Agent Cost C_A($)	181,152	*User Cost C_U($)*	6,796
Total Cost($)			187,948
Lane-mile/Period			3.13
# of needed Periods			3.84(≈4)
Maximal Queue Length (mile)			0

(d) Optimization Result of Scenario 4 (Low Traffic Level, Low Intensity Work)

Zone No.	***Starting Time***	***Ending Time***	***Duration***	***Length***	***Str#1***	***Str#2***	***Str#3***	***Str#4***
(#)	(0-24)	(0-24)	(hr)		Lane Closure	Work Rate	Detour	Merge Control
1	20:00	07:00	11	1.4	Double	Normal	No	No
2	07:00	14:00	7	1.25	Single	Normal	No	Yes

C_M($)	109,292	*C_D($)*	21,150
C_S($)	1,329	*C_V($)*	1,704
C_I($)	12,000	*C_E($)*	2,448
Agent Cost C_A($)	122,621	*User Cost C_U($)*	25,302
Total Cost($)			147,923
Lane-mile/Period			4.06
# of needed Periods			2.95(≈3)
Maximal Queue Length (mile)			0.32

When traffic level is high (Scenarios 1 and 3), night-time double lane closure is the best option to avoid disturbing heavy traffic in daytime. When conducting high intensity maintenance work (Scenario 1), the additional cost required to accelerate the work (10%-20% of normal unit cost) is too high to be compensated by the resulting user cost savings and therefore a normal work rate is more cost-effective. However, for low intensity work (Scenario 3), it is worthwhile to increase the work rate since user cost saving now can easily justify the relatively low additional cost.

Under low traffic conditions (Scenarios 2 and 4), the optimal strategy is to schedule a single lane closure in daytime and a double lane closure at night. Thanks to the increased work zone capacity obtained by employing an advanced merge control system, a single lane closure during the morning peak hour would not cause unacceptable queuing delay and thus only one work break is scheduled during the afternoon peak period. Since there is plenty time to perform maintenance work, it is unnecessary to spend additional money on accelerated construction. An interesting finding is that in Scenario 2 (low traffic level and high work intensity) two subsequent short work zones are more preferable than one long work zone in daytime window. This occurs because the average cost per lane mile is reduced by putting more work at nighttime window although more periods are needed to complete the whole project. The result indicates that when idling cost or value of work time does not play an important role and the fixed time/cost to set up a work zone are relatively low, multiple sub-zones strategy may outperform single work zone strategy with respect to average cost per lane mile.

(2) Optimality Analysis

The Maryland State Highway Administration (SHA) lane-closure policies for highway maintenance (*Chen, 2003*) are 9:00 a.m. - 3:00 p.m. and 7:00 p.m. - 5:00 a.m. for single-lane closure; 10:00 p.m. -5:00 a. m. for two-lane closure; and 12:00 a.m. – 5:00 a.m. for three-lane closure. To demonstrate the effectiveness of the optimization model, we compare the optimized work zone management plans with the following three conventional policies in all four scenarios:

C1: 7:00 p.m-5:00 a.m. single lane closure, without deploying any additional strategy.

C2: 10:00 p.m.-5:00 a.m. two-lane closure, without deploying any additional strategy.

C3: 9:00 a.m. – 3:00 p.m. single lane closure and 10:00 p.m.-5:00 a.m. two-lane closure, without deploying any additional strategy.

As reflected in Figure 4-21, the optimized work zone management plan t outperforms the above three conventional policies with respect to the total work zone cost in all four test scenarios. This comparison result clearly indicates the fact that the maintenance project can be accomplished more cost-efficiently by improving lane closure plan and investing on proper work zone impact management strategies.

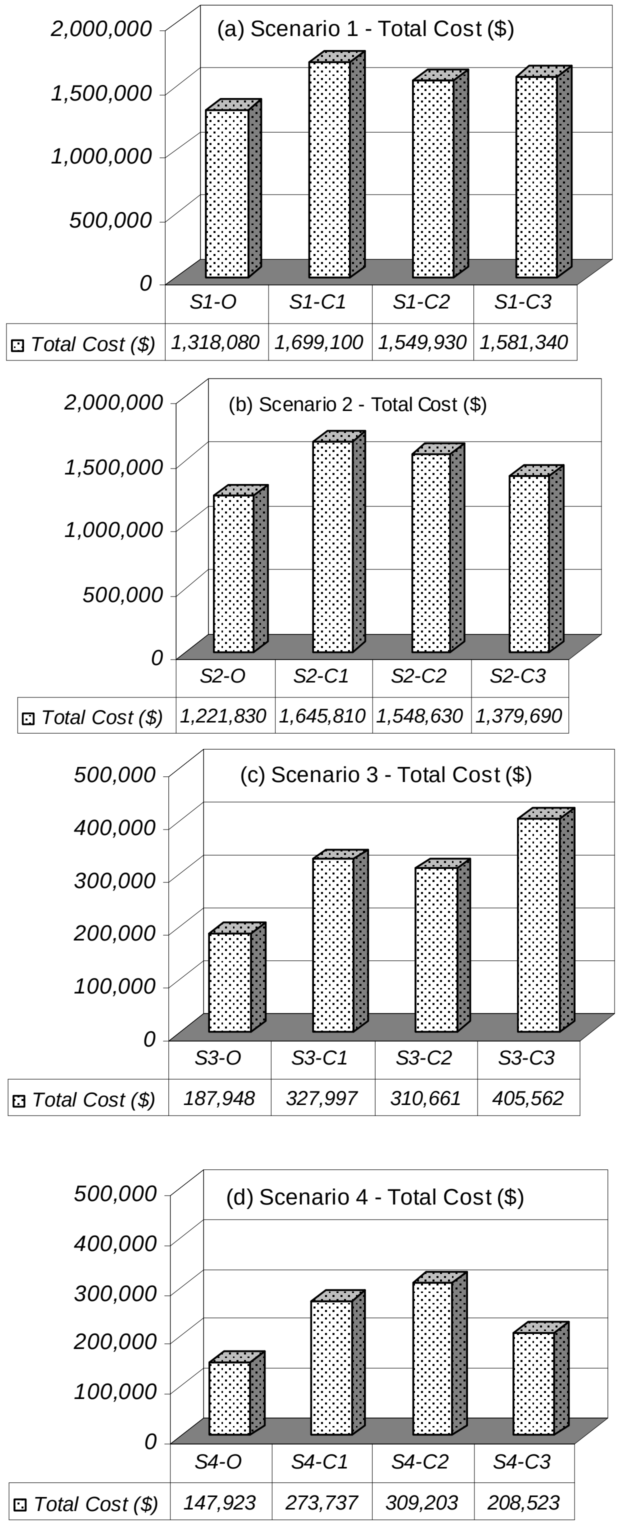


Figure 4-21 Comparison of Optimized Results and Conventional Policies

(3) Reliability Analysis

Since the proposed two-stage simulated annealing algorithm is a stochastic method, a hundred replications with different random number seeds are performed to optimize the work zone plan for each test scenario to verify the reliability of the optimization model under different circumstances. The number of generations is set to 50 at the first stage and 200 at the second stage while population size is set to 200 and 20 at the first and second stages, respectively. The minimized total costs over 100 replications are illustrated in Figure 4-22 and the statics are provided in Table 4-9.

It can be seen that the optimized result is quite reliable in scenarios 1 and 3 with high traffic level. The key reason is that the time windows which permit lane closure is fairly limited, which significantly reduces the space of feasible solutions. The coefficient of variation (CV) increases when traffic level decreases in scenarios 2 and 4 because with less constraint on working time, the possible number of work zones increases and hence the solution space becomes much larger considering the different combinations of decision variables for each work zone. Either for scenario 2 or scenario 4, the CV is below 1%. By checking the details of the optimized results over 100 replications, we found that most of them have the same number of work zones and the management strategies selected for each work zone are also almost same. The slight variation of their total cost comes from the small change of the starting time or ending time of one or more sub work zones.

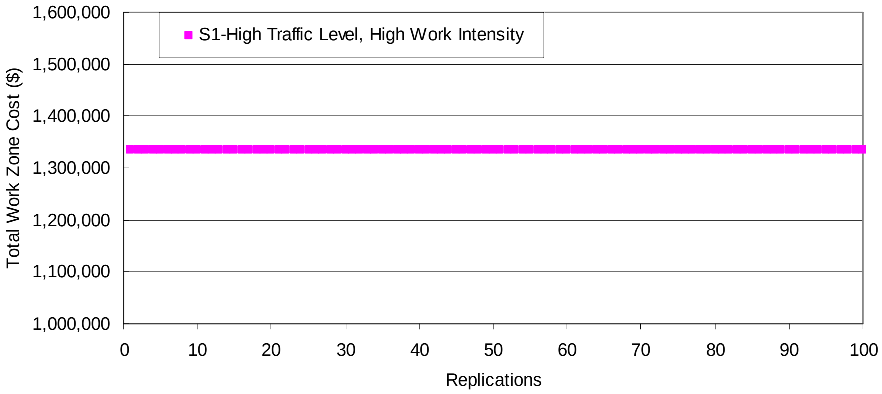
The optimality and reliability analysis proves that the proposed optimization algorithm is reliable to obtain near-optimal solutions. It is recommended to use the best result

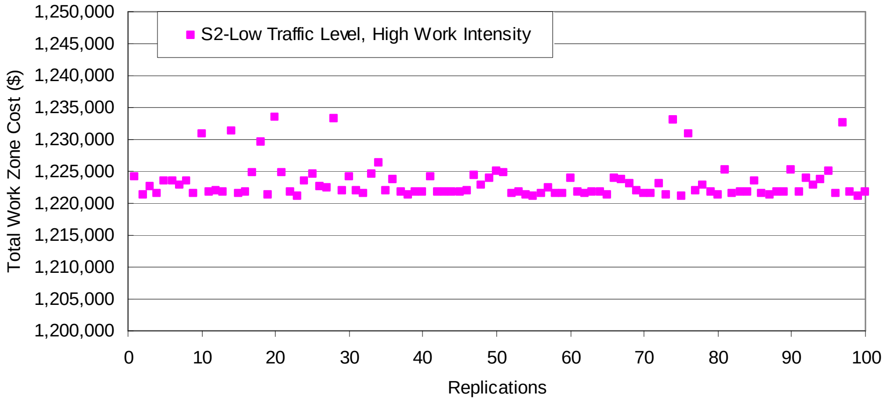
obtained from multiple replications to reduce the effect of stochastic feature. Note that the solution quality can also be improved by increasing the population size at the first stage and the number of generation at the second stage, especially for problems with a large solution space.

Table 4-9 Statistics of the Optimized Total Cost

Scenario	Rep.	Mean	Min	Max	STD*	CV*
1	100	1,334,880	1,334,880	1,334,880	0	0.0%
2	100	1,223,251	1,221,170	1,233,380	2,838	0.2%
3	100	208,765	208,765	208,765	0	0.0%
4	100	148,661	147,923	152,092	1,322	0.9%

*STD: Standard deviation.
*CV: Coefficient of Variation, defined as the ratio of the standard deviation to the mean.





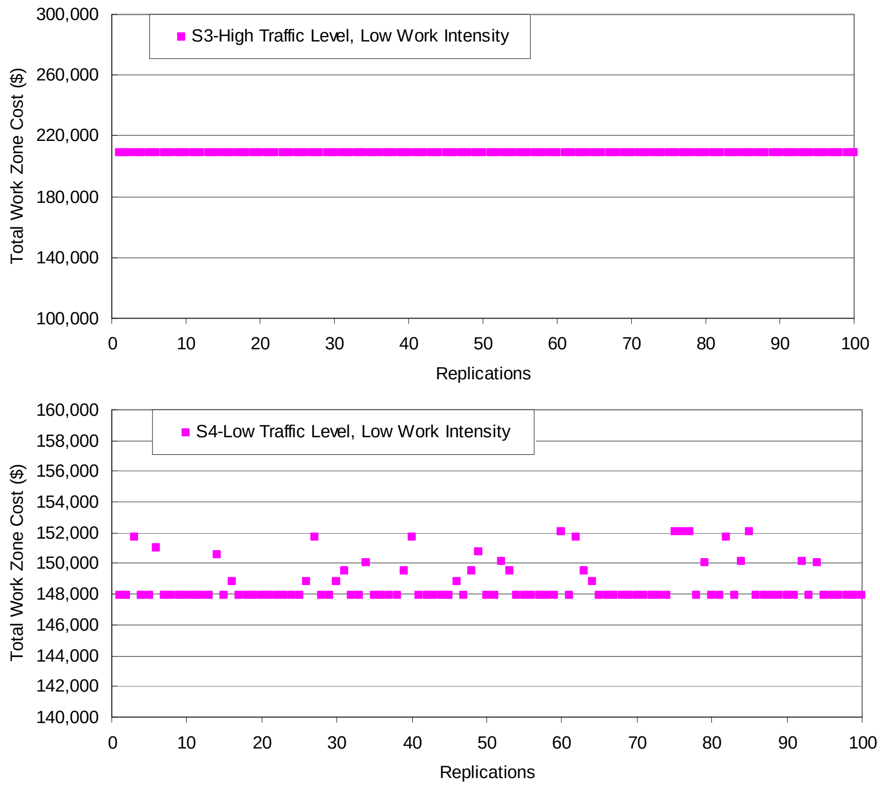


Figure 4-22 Minimized Total Costs over 100 Replications in Four Scenarios

4.6.4 Sensitivity Analysis

In this subsection, we seek to examine the impact of key input parameters on the optimized work zone management plan. Scenarios with varying traffic level, detour policy, idling cost, ,project deadline, and fixed work zone setup time are tested because they are critical factors in the design of work zone management plan from the transportation agencies' and maintenance contractors' points of view. The result variation affected by different analysis period (e.g. weekday vs. weekend) and user behavior model (e.g. System Optimization Model vs. Route Choice Model vs. User Equilibrium Model) are also investigated in this test.

For all test scenarios, the population sizes at the two stages are set at 200 and 20, respectively, and the numbers of generations are set at 50 and 200, respectively. The

default cyclic period is a typical weekday (24 hours). Each scenario is re-optimized in ten replications and the analysis is based on the best solution obtained.

4.6.4.1 Traffic Volume and Detour Policy

Ten Traffic levels with traffic volume multipliers ranging from 0.2 to 2.0 for mainline traffic volumes are tested. For each traffic level, the distribution of the traffic flow stays unchanged while the hourly volume increases or decreases to the product of the baseline volume and the traffic level multiplier.

Table 4-10 Traffic Levels Ranging from 0.2 to 2.0

Traffic Level	**0.2**	**0.4**	**0.6**	**0.8**	**1.0***	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
Q1 AADT	19,862	39,725	59,588	79,451	99,314	119,176	139,039	158,902	178,765	198,628
Q2 AADT	19,910	39,820	59,731	79,641	99,552	119,462	139,372	159,283	179,193	199,104

*Baseline: Traffic Level =1.0

To investigate the impacts of detour policy, the optimized results while employing the following four detour policies are analyzed:

- No detour control: all traffic stays on the mainline route.
- ITS provided to guide road users to use detour route. Assuming fully compliance rate, the time-varying detour fraction is determined with the system optimization (SO) model.
- Road users respond to the ITS detour control system. The time-varying detour fraction is determined with the Route Choice (RC) model.
- Road users respond to the ITS detour control system. The time-varying detour fraction is determined with the User Equilibrium (UE) model.

(1) No Detour Control

Without applying any detour control strategy, the optimized solutions in traffic level ranging from 0.2 to 1.8 are displayed in

Table 4-11. As the traffic increases, the lane closure time during one period decreases from 24 hours per day to 7.5 hours per day and the number of periods needed to complete the maintenance work increases from 4 days to 14 days in order to keep the impact of the work zone activity on motorists at an acceptable level. It can be seen that optimized lane closure time window and associated lane closure type significantly change with the traffic level:

- At traffic levels 0.2-0.4, two lanes can be closed the whole day while still providing enough road capacity for mainline traffic. At level 0.2, one long work zone with 24-hour double lane closure is the best choice. At level 0.4, it is better to set up two 12-hour double lane closure work zones because shorter work zone length reduces the user moving delay and the resulting user cost saving exceeds the additional cost of setting up one more work zone.
- At traffic levels 0.6-0.8, traffic volumes in morning and afternoon peak-hours exceed the reduced capacity. Optimized work zones are scheduled in daytime and nighttime off-peak time windows to avoid extensive queuing delays. During daytime off-peak hours, double lane closure is still affordable at level 0.6 while single lane closure is a better choice at level 0.8. Note that at traffic

levels above 0.4, merge control (strategy type 4) is always preferable because the increased work zone capacity deserves its additional cost.

- At traffic levels 1.0-2.0, lane closures are acceptable only during nighttime hours. The duration of the double lane nighttime work zones keeps decreasing when traffic level increases even with the help of the merge control strategy. After the traffic level exceeds 1.8, it becomes beneficial to accelerate the maintenance work by adopting the highest productivity rate.

Table 4-11 Optimization Solutions at varying traffic levels without Detour Control

Traffic Level	***# of periods****	***Work Time***	***Zones***	***Str#1***	***Str#2***	***Str#3***	***Str#4***	***C_A***	***C_U***	***C_T***
	#	hr/ period	#	Lane Closure	Work Rate	Detour	Merge Control	$	$	$
0.2	4	24	16:00-16:00	double	normal	no	no	1,059,490	8,024	1,067,514
0.4	4	24	16:00-04:00	double	normal	no	yes	1,069,056	17,691	1,086,747
			04:00-16:00	double	normal	no	yes			
0.6	6	17	19:00-07:30	double	normal	no	yes	1,134,023	23,349	1,157,372
			09:30-14:00	double	normal	no	yes			
0.8	7	15.5	20:00-07:00	double	normal	no	yes	1,213,767	17,645	1,231,413
			09:00-13:30	single	normal	no	yes			
1.0	10	10	20:30-06:30	double	normal	no	yes	1,295,360	22,719	1,318,079
1.2	12	8.5	21:30-06:00	double	normal	no	yes	1,384,023	10,768	1,394,791
1.4	13	8	22:00-06:00	double	normal	no	yes	1,427,200	9,773	1,436,973
1.6	14	7.5	22:30-06:00	double	normal	no	yes	1,472,593	18,683	1,491,276
1.8	12	6.5	23:00-05:30	double	fast	no	yes	1,634,227	10,362	1,644,589
2.0	16	5.5	23:30-05:00	double	fast	no	yes	1,806,320	6,403	1,812,723

*Duration of a cyclic period=24 hr

(2) Detour Control

(2.1) Detour Control-System Optimization (SO) Model

Employing an advanced detour control system, the optimized work zone plans at different traffic levels are listed in Table 4-12. The time-varying detour fractions achieving system optimization are provided in Table 4-13and Figure 4-23.

Table 4-12 Optimization Solutions at varying traffic levels with Detour Control (SO)

Traffic Level	***# of periods ****	***Work Time***	***Zones***	***Str#1***	***Str#2***	***Str#3***	***Str#4***	***C_A***	***C_U***	***C_T***
	#	hr/ period	#	Lane Closure	Work Rate	Detour	Merge Control	$	$	$
0.2	4	24	16:00-16:00	double	normal	SO	no	1,077,990	8,024	1,086,020
0.4	4	24	16:00-16:00	double	normal	SO	yes	1,082,530	23,198	1,105,730
0.6	5	21	19:00-09:00	double	normal	SO	no	1,108,560	41,707	1,150,270
			09:00-16:00	double	normal	SO	yes			
0.8	5	18	20:00-14:00	double	normal	SO	yes	1,121,280	76,059	1,197,340
1	7	13	19:00-08:00	double	normal	SO	yes	1,199,860	44,203	1,244,070
1.2	8	12	19:00-07:00	double	normal	SO	yes	1,235,330	43,927	1,279,250
1.4	9	11	20:00-07:00	double	normal	SO	yes	1,252,080	34,563	1,286,640
1.6	10	10	20:00-06:00	double	normal	SO	yes	1,319,360	36,594	1,355,950
1.8	11	9	21:00-06:00	double	normal	SO	yes	1,362,210	27,878	1,390,088
2	11	9	21:00-06:00	double	normal	SO	yes	1,368,240	34,871	1,403,110

Table 4-13 Time-varying Detour Fraction at varying traffic levels with Detour Control (SO)

Traffic Level	**0.2**	**0.4**	**0.6**	**0.8**	**1**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0:00	0	0	0	0	0	0	0	0	0	0
1:00	0	0	0	0	0	0	0	0	0	0
2:00	0	0	0	0	0	0	0	0	0	0
3:00	0	0	0	0	0	0	0	0	0	0
4:00	0	0	0	0	0	0	0	0	0	0
5:00	0	0	0	0	0	0	0	0.1	0.2	0.28
6:00	0	0	0	0.04	0.22	0.36	0.44	0	0	0
7:00	0	0	0.3	0.38	0.46	0	0	0	0	0
8:00	0	0	0.36	0.42	0	0	0	0	0	0
9:00	0	0	0.12	0.36	0	0	0	0	0	0
10:00	0	0	0	0.26	0	0	0	0	0	0
11:00	0	0	0.02	0.26	0	0	0	0	0	0
12:00	0	0	0.04	0.28	0	0	0	0	0	0
13:00	0	0	0.04	0.28	0	0	0	0	0	0
14:00	0	0	0.2	0	0	0	0	0	0	0
15:00	0	0	0.3	0	0	0	0	0	0	0
16:00	0	0.02	0	0	0	0	0	0	0	0
17:00	0	0	0	0	0	0	0	0	0	0
18:00	0	0	0	0	0	0	0	0	0	0
19:00	0	0	0.2	0	0.44	0.48	0	0	0	0
20:00	0	0	0	0.04	0.22	0.4	0.44	0.48	0	0
21:00	0	0	0	0	0.02	0.18	0.3	0.4	0.44	0.48
22:00	0	0	0	0	0	0	0.12	0.22	0.32	0.38
23:00	0	0	0	0	0	0	0	0	0.08	0.18

As the mainline traffic increases, the optimal lane closure policy changes from whole day double lane closure, to double lane closures in off-peak hours, and then to nighttime double closures. By diverting a system-optimized fraction of mainline traffic to the alternative route, spare capacity in the network is fully utilized and the remaining traffic volumes on the mainline route can be significantly reduced. As a result, work zones with longer duration are allowed compared to the results without detour control. Merge control is desirable as long as mainline capacity is still limited. At high traffic levels, detour control strategy is effective and economic and there is no need to apply for costly work acceleration.

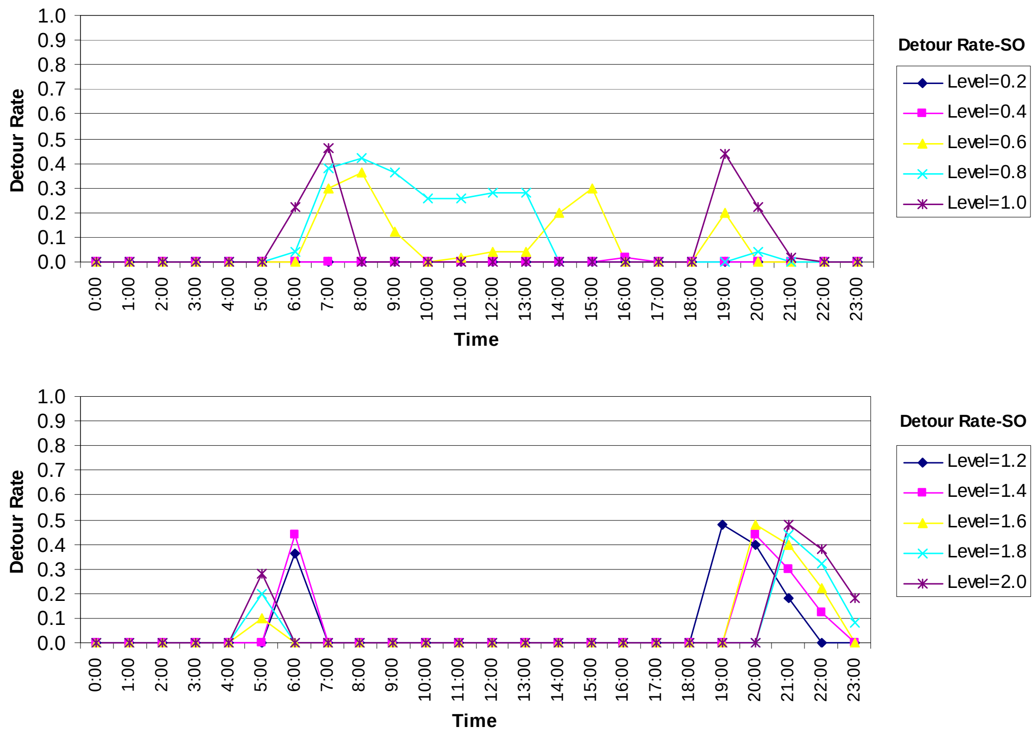


Figure 4-23 Detour Fraction at varying traffic levels (SO)

(2.2) Detour Control-Route Choice (RC) Model

In this test, the advanced detour control system is employed to provide travel time information on mainline and detour routes. It is assumed that road users, as independent decision makers, will choose whether or not to detour based on their acceptance of travel time difference. A time-varying diversion fraction is derived from the Logit-based choice model introduced in Chapter 3. Table 4-14 lists the optimized solutions. Table 4-15 and Figure 4-24 provide the detour fraction information.

Table 4-14 Optimization Solutions at varying traffic levels with Detour Control (RC)

Traffic Level	***# of periods****	***Work Time***	***Zones***	***Str#1***	***Str#2***	***Str#3***	***Str#4***	C_A	C_U	C_T
	#	hr /period	#	Lane Closure	Work Rate	Detour	Merge Control	$	$	$
0.2	4	24	16:00-16:00	double	normal	RC	no	1,077,990	8,024	1,086,020
0.4	4	24	16:00-16:00	double	normal	RC	yes	1,082,530	23,069	1,105,600
0.6	5	17.5	19:30-13:00	double	normal	RC	no	1,119,770	53,013	1,172,790
0.8	6	17	19:00-08:00	double	normal	RC	yes	1,158,010	57,719	1,215,730
			09:00-13:00	double	normal	RC	yes			
1	7	12	19:00-07:00	double	normal	RC	yes	1,235,330	31,833	1,267,160
1.2	8	12	19:00-07:00	double	normal	RC	yes	1,235,330	59,255	1,294,580
1.4	9	11	20:00-07:00	double	normal	RC	yes	1,275,120	41,431	1,316,550
1.6	10	10	20:00-07:00	double	normal	RC	yes	1,275,120	64,758	1,339,880
1.8	11	9	20:30-05:30	double	normal	RC	yes	1,368,240	46,185	1,414,420
2	11	9	21:00-06:00	double	normal	RC	yes	1,362,210	48,811	1,411,020

Table 4-15 Time-varying Detour Fraction at varying traffic levels with Detour Control (RC)

Traffic Level	**0.2**	**0.4**	**0.6**	**0.8**	**1**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0:00	0	0	0	0	0	0	0	0	0	0
1:00	0	0	0	0	0	0	0	0	0	0
2:00	0	0	0	0	0	0	0	0	0	0
3:00	0	0	0	0	0	0	0	0	0	0
4:00	0	0	0	0	0	0	0	0	0	0
5:00	0	0	0	0	0	0	0	0	0	0.55
6:00	0	0	0	0	0.53	0.59	0.6	0.59	0	0
7:00	0	0	0.53	0.55	0	0	0	0	0	0
8:00	0	0	0.55	0	0	0	0	0	0	0
9:00	0	0	0.53	0.54	0	0	0	0	0	0
10:00	0	0	0	0.53	0	0	0	0	0	0
11:00	0	0	0.53	0.53	0	0	0	0	0	0
12:00	0	0	0	0.53	0	0	0	0	0	0
13:00	0	0	0	0	0	0	0	0	0	0
14:00	0	0	0	0	0	0	0	0	0	0
15:00	0	0	0	0	0	0	0	0	0	0

16:00	0	0	0	0	0	0	0	0	0	0
17:00	0	0	0	0	0	0	0	0	0	0
18:00	0	0	0	0	0	0	0	0	0	0
19:00	0	0	0	0.53	0.6	0.57	0	0	0	0
20:00	0	0	0	0	0.53	0.6	0.6	0.6	0.52	0
21:00	0	0	0	0	0	0.53	0.53	0.55	0.62	0.61
22:00	0	0	0	0	0	0	0	0.53	0.53	0.52
23:00	0	0	0	0	0	0	0	0	0	0.53

Comparison between Table 4-12 and Table 4-14 shows the optimal work zone plans with RC detour model are similar to those with SO detour model, with slightly shorter work zone durations at traffic levels 0.6-1.0.

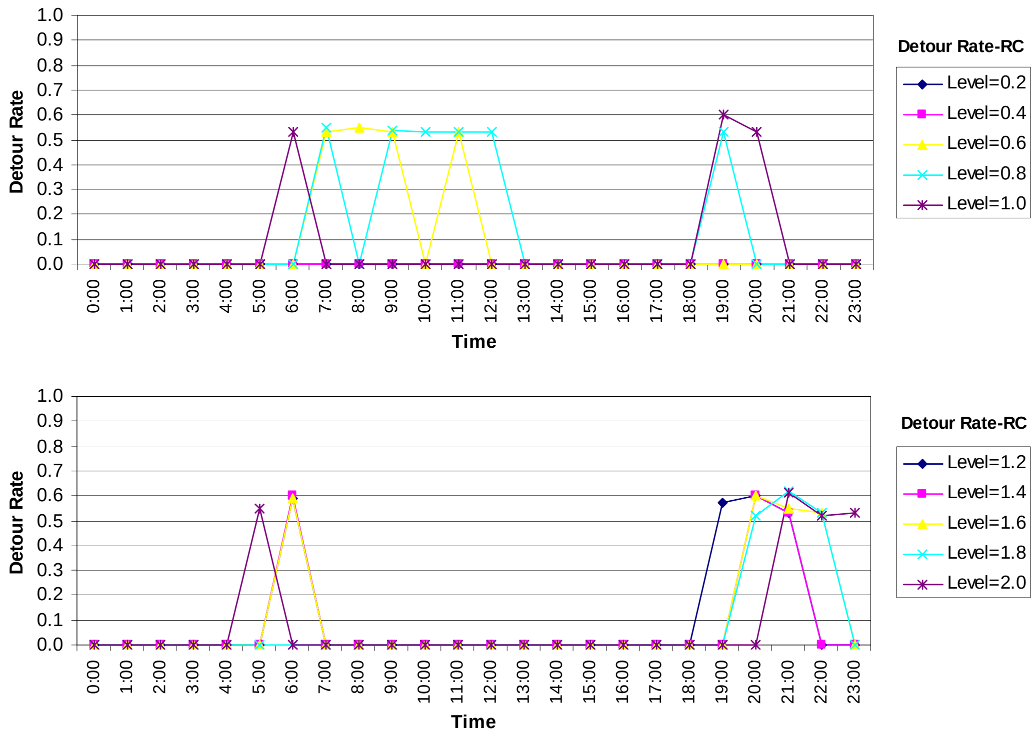


Figure 4-24 Detour Fraction at varying traffic levels (RC)

(2.3) Detour Control-User Equilibrium (UE) Model

This test assumes that travelers have good knowledge of the traffic condition resulting from work zones and user equilibrium can be achieved when detour control system is employed. The optimized work zone plans shown in Table 4-16 follow the same trend

as those in SO condition but the work zone durations are shorter at traffic levels 1-1.8. Table 4-17 and Figure 4-25 provide the time-varying diversion fractions derived from the UE model.

Table 4-16 Optimization Solutions at varying traffic levels with Detour Control (UE)

Traffic Level	# of periods *	Work Time	Zones	Str#1	Str#2	Str#3	Str#4	C_A	C_U	C_T
	#	hr/ period	#	Lane Closure	Work Rate	Detour	Merge Control	$	$	$
0.2	4	24	16:00-16:00	double	normal	UE	no	1,077,990	8,024	1,086,020
0.4	4	24	16:00-16:00	double	normal	UE	yes	1,082,530	23,069	1,105,600
0.6	5	17	19:30-07:30	double	normal	UE	yes	1,160,010	23,349	1,183,360
			09:30-14:00	double	normal	UE	yes			
0.8	8	12	19:30-07:30	double	normal	UE	yes	1,235,330	28,252	1,263,580
1	10	10	20:30-06:30	double	normal	UE	yes	1,319,360	20,257	1,339,620
1.2	11	9	21:30-06:30	double	normal	UE	yes	1,368,240	17,858	1,386,100
1.4	12	8.5	22:00-06:30	double	normal	UE	yes	1,410,010	18,996	1,429,010
1.6	14	7.5	22:30-06:00	double	normal	UE	yes	1,500,520	17,684	1,518,210
1.8	14	6.5	23:00-05:30	double	normal	UE	yes	1,625,630	10,259	1,635,890

Table 4-17 Time-varying Detour Fraction at varying traffic levels with Detour Control (RC)

Traffic Level	0.2	0.4	0.6	0.8	1	1.2	1.4	1.6	1.8	2.0
0:00	0	0	0	0	0	0	0	0	0	0
1:00	0	0	0	0	0	0	0	0	0	0
2:00	0	0	0	0	0	0	0	0	0	0
3:00	0	0	0	0	0	0	0	0	0	0
4:00	0	0	0	0	0	0	0	0	0	0
5:00	0	0	0	0	0	0	0	0	0	0.22
6:00	0	0	0	0	0.02	0.2	0.3	0	0	0
7:00	0	0	0	0.24	0	0	0	0	0	0
8:00	0	0	0	0	0	0	0	0	0	0
9:00	0	0	0	0	0	0	0	0	0	0
10:00	0	0	0	0	0	0	0	0	0	0
11:00	0	0	0	0	0	0	0	0	0	0
12:00	0	0	0	0	0	0	0	0	0	0
13:00	0	0	0	0	0	0	0	0	0	0
14:00	0	0	0	0	0	0	0	0	0	0
15:00	0	0	0	0	0	0	0	0	0	0
16:00	0	0	0	0	0	0	0	0	0	0
17:00	0	0	0	0	0	0	0	0	0	0
18:00	0	0	0	0	0	0	0	0	0	0
19:00	0	0	0	0.12	0	0	0	0	0	0
20:00	0	0	0	0	0.04	0	0	0	0	0

21:00	0	0	0	0	0	0	0	0	0	0
22:00	0	0	0	0	0	0	0	0.02	0	0
23:00	0	0	0	0	0	0	0	0	0	0.1

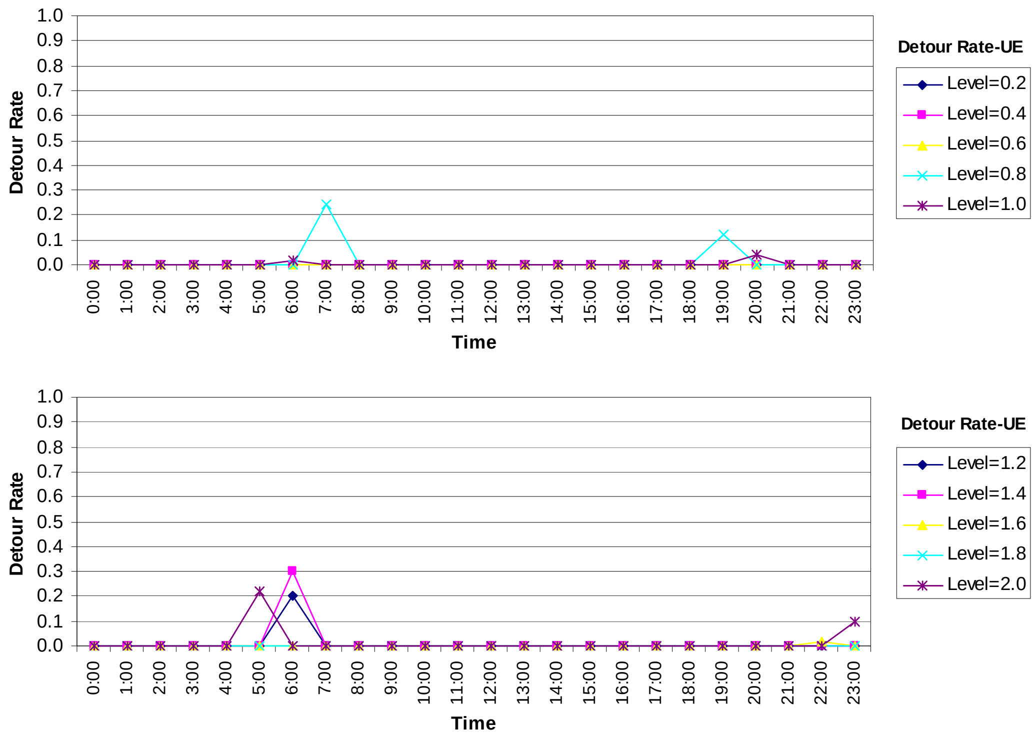


Figure 4-25 Detour Fraction at varying traffic levels (UE)

(3) Comparison of Detour Fractions Derived from Different Detour Models

Fair comparison of different detour models has to be based on the same work zone characteristics and traffic conditions. The work zone management plan in Table 4-18 is evaluated four times. Each time a different detour model is used to derive time-varying detour fractions. The derived detour fractions are displayed in Figure 4-26. It can be seen that UE detour fractions are lower for UE than for SO detour. This may occur because in this case study the detour route is relatively long and has a lower free flow speed compared to mainlines (5.16 miles vs. 3.1 miles; 47 mph vs. 65 mph). UE has been reached even though there is still spare capacity on the detour. The RC model yields the highest detour fractions. A possible reason is that travelers whose behavior is

described by the choice model are quite sensitive to the travel time difference and more travelers would choose to detour with less consideration of the new traffic conditions that their detour may result.

Table 4-18 Input parameters of test work zone plan

Traffic Level	***# of periods****	***Work Time***	***Zones***	***Str#1***	***Str#2***	***Str#3***	***Str#4***
	#	hr /period	#	Lane Closure	Work Rate	Detour	Merge Control
1	7	13	19:00-08:00	double	normal	-	yes

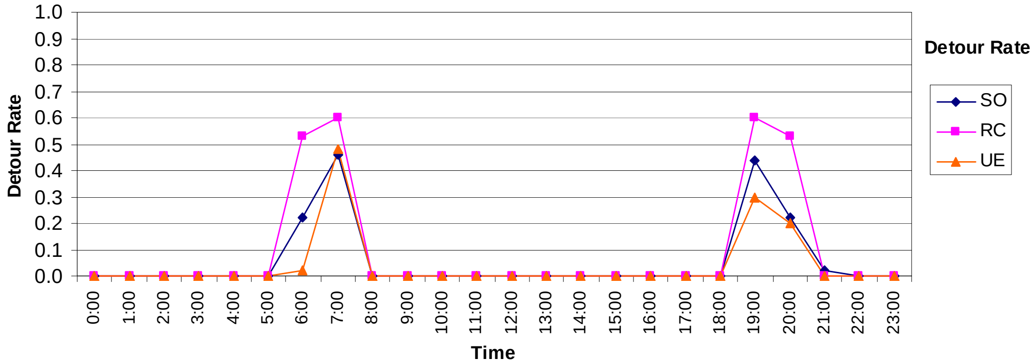


Figure 4-26 Detour Fraction at varying traffic levels (UE)

The costs associated the test work zone plan with different detour models are shown in Table 4-19. Work zone plans with any detour control can outperform the same operation plan without detour control in terms of the total costs because user costs saving due to reduced queuing delay compensate for the additional agency costs spent on the control system. The work zone plan with SO detour control achieves the lowest total cost since the traffic assignment is optimized from a system point of view.

Table 4-19 Comparison of Detour Models based on the Test Work Zone Plan

	C_M	C_S	C_I	C_A	C_D	C_V	C_E	C_U	C_T
N/A	1,062,980	5,236	110,000	1,178,220	320,076	18,629	27,722	366,427	1,544,650
SO	1,062,980	26,880	110,000	1,199,860	22,727	20,268	1,209	44,203	1,244,070
RC	1,062,980	26,880	110,000	1,199,860	28,136	27,610	547	56,294	1,256,160
UE	1,062,980	26,880	110,000	1,199,860	67,516	16,246	5,931	89,694	1,289,560

(4) Comparison of Detour Fractions Derived from Different Detour Models

Figure 4-27 (a), (b) and (c) shows the agency cost, user cost, and total cost of the optimized work zone management plan under different detour conditions. Employing detour control can allow work zones with longer duration and larger space, which can greatly reduce agency costs, thus compensating for the increased user cost.

Among the four detour models, the SO model can achieve the lowest cost. In practice, SO detour fractions are treated more as a control objective. Although flashing Dynamic Message Sign with a specific frequency may help controlling the diversion fraction, it is difficult to obtain the exact SO result due to diversity and uncontrollability of actual road user behaviors. The RC model also yields low work zone cost. It has to be noted that the parameters of route choice model should be estimated based on data from stated-preference surveys or other user behavior studies. Overly optimistic estimation of detour rates may lead to costly work zone decisions. The benefit of employing detour control with UE model is not obvious. The major reason is that the user cost saving cannot outweigh the additional traffic control costs. Its performance may be improved when the detour route is more attractive (e.g. shorter length, higher speed, or higher capacity) and the control cost is lower.

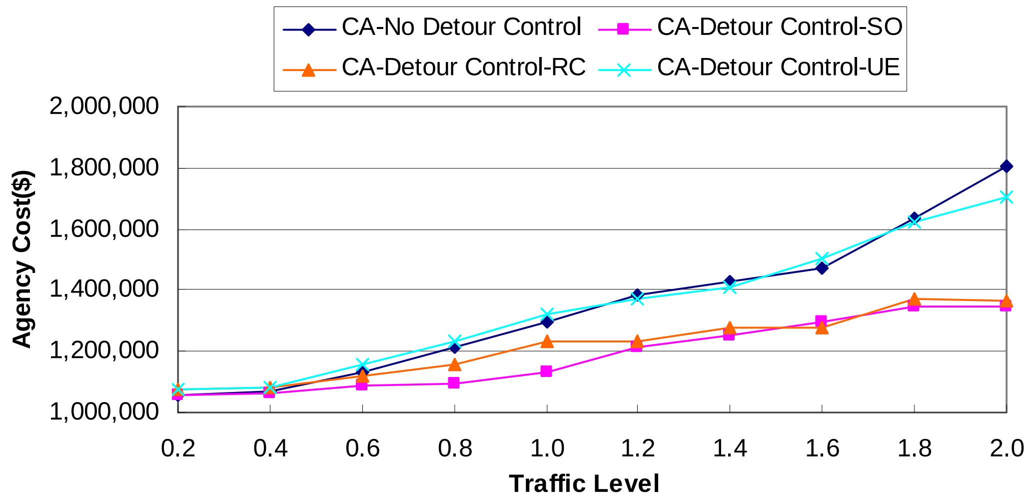
4.6.4.2 Idling Cost

For the baseline scenario (Scenario 1), sensitivity analysis is conducted by increasing the idling cost from 1000$/hr to 9000 $/hr. When the detour is not available, night-time double lane closure turns out to be the optimal solution. With the idling cost increases, a faster work rate is desirable to decrease the total idling cost by reducing the total time

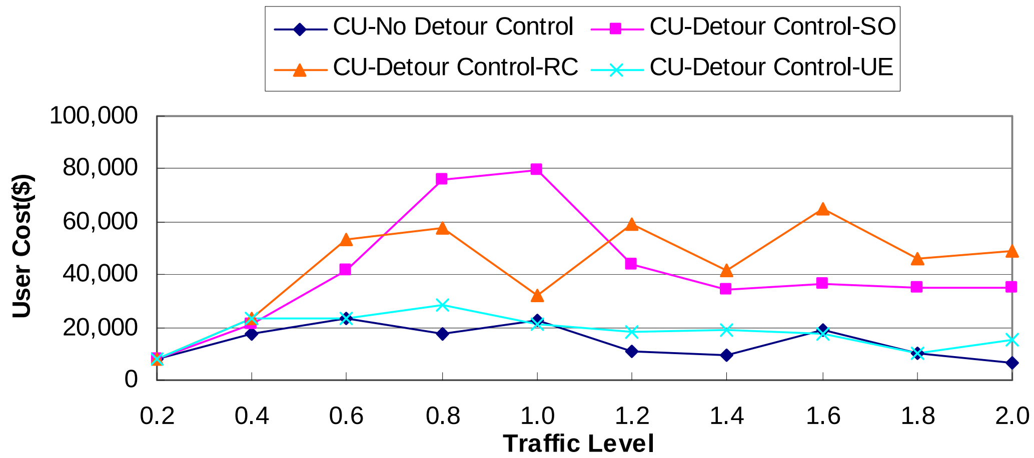
required to complete the maintenance work (Table 4-20). If detour control with the SO detour model is employed, accelerating construction is not preferable and total project time can be more economically reduced by scheduling one more single lane closure in day-time off-peak hours (Table 4-21). The costs information is provided in Figure 4-28.

4.6.4.3 Project Deadline

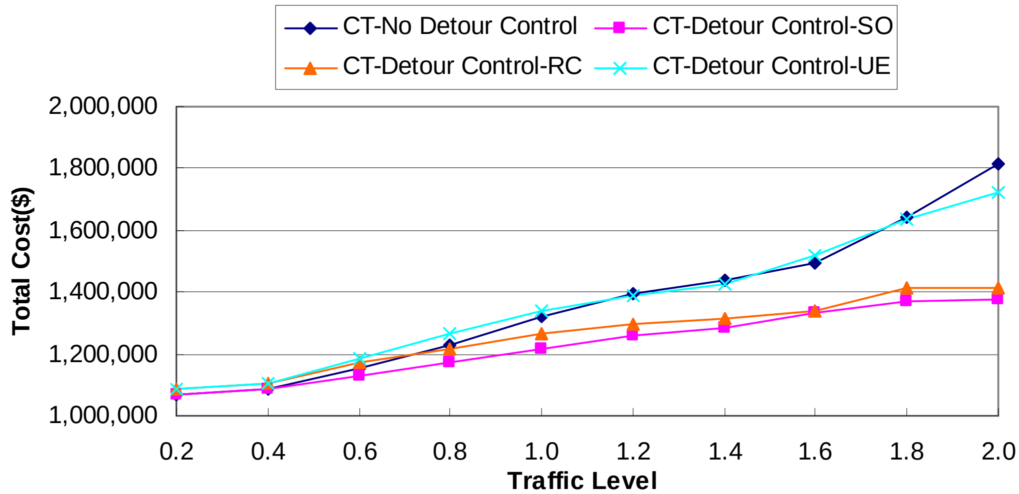
The maximum allowable number of periods for completing the maintenance project (project deadline) is a constraint in the work zone optimization model. Optimization results with decreasing project deadline with and without detour control are listed in Table 4-22 and Table 4-23. It can be seen that setting up a single lane closure work zone in daytime off-peak hours, applying faster work rate, and diverting traffic to alternative route can be employed together to satisfy a tight project deadline. This may greatly increase both agency cost and user cost due to additional costs spent on those management strategies and to severe traffic interruption. Figure 4-29 illustrates the variation of work zone costs with decreasing project deadlines. When the maximum allowable number of periods exceeds 10 without detour control or exceeds 6 with detour control, the optimal work zone management plan will remain unchanged. When the maximum allowable number of periods gets below 8 without detour control and below 5 with detour control, the work zone costs will dramatically rise.



(a) Comparison of Agency Costs



(b) Comparison of User Costs



(c) Comparison of Total Costs

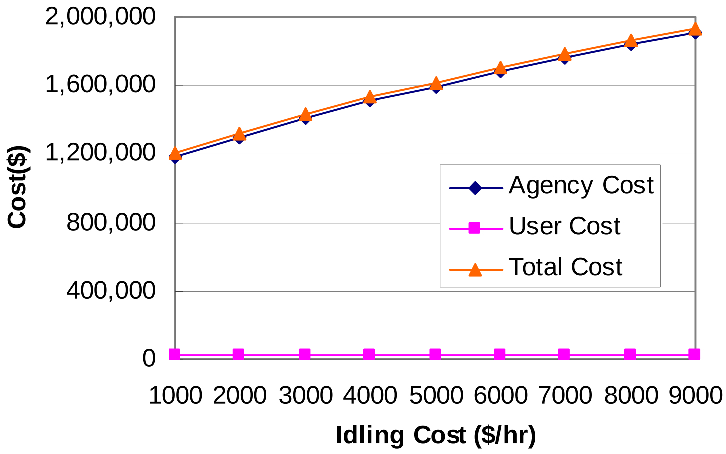
Figure 4-27 Comparison of Optimized Results with and without Detour Control Strategy

Table 4-20 Optimal Solutions with Varying Idling Cost (No Detour)

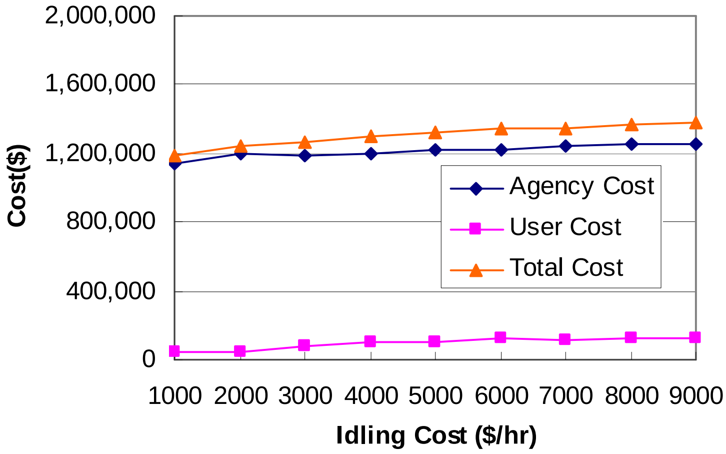
Idling Cost	# of periods *	Work Time	Zones	Str#1	Str#2	Str#3	Str#4	CA	CU	CT
$/hr	#	hr/ period	#	Lane Closure	Work Rate	Detour	Merge Control	$	$	$
1000	10	10	20:30-06:30	double	normal	no	yes	1,183,360	22,719	1,206,079
2000	10	10	20:30-06:30	double	normal	no	yes	1,295,360	22,719	1,318,079
3000	10	10	20:30-06:30	double	normal	no	yes	1,407,360	22,719	1,430,079
4000	8	10	20:30-06:30	double	medium	no	yes	1,509,888	23,078	1,532,966
5000	8	10	20:30-06:30	double	medium	no	yes	1,593,888	23,078	1,616,966
6000	8	10	20:30-06:30	double	medium	no	yes	1,677,888	23,078	1,700,966
7000	8	10	20:30-06:30	double	fast	no	yes	1,761,888	23,078	1,784,966
8000	7	10	20:30-06:30	double	fast	no	yes	1,837,952	23,335	1,861,287
9000	7	10	20:30-06:30	double	fast	no	yes	1,907,952	23,335	1,931,287

Table 4-21 Optimal Solutions with Varying Idling Cost (Detour Control-SO)

Idling Cost	# of periods *	Work Time	Zones	Str#1	Str#2	Str#3	Str#4	CA	CU	CT
$/hr	#	hr/ period	#	Lane Closure	Work Rate	Detour	Merge Control	$	$	$
1000	7	13	19:00-08:00	double	normal	SO	yes	1,144,860	44,203	1,189,070
2000	7	13	19:00-08:00	double	normal	SO	yes	1,199,860	44,203	1,244,070
3000	6	17	20:00-08:00	double	normal	SO	yes	1,187,010	81,880	1,268,890
			09:00-14:00	double	normal	SO	yes			
4000	6	18	19:00-08:00	double	normal	SO	yes	1,198,240	96,452	1,294,690
			09:00-14:00	double	normal	SO	yes			
5000	6	18	19:00-08:00	double	normal	SO	yes	1,223,240	96,452	1,319,690
			09:00-14:00	double	normal	SO	yes			
6000	6	19	19:00-08:00	double	normal	SO	yes	1,222,700	120,384	1,343,090
			09:00-15:00	double	normal	SO	yes			
7000	6	20	19:00-08:00	double	normal	SO	yes	1,237,840	110,894	1,348,740
			08:00-11:00	single	normal	SO	yes			
			11:00-15:00	double	normal	SO	yes			
8000	5	21	19:00-08:00	double	normal	SO	yes	1,249,280	122,077	1,371,350
			08:00-16:00	single	normal	SO	yes			
9000	5	21	19:00-08:00	double	normal	SO	yes	1,258,280	122,077	1,380,350
			08:00-16:00	single	normal	SO	yes			



(a) No Detour Control



(b) Detour Control-SO

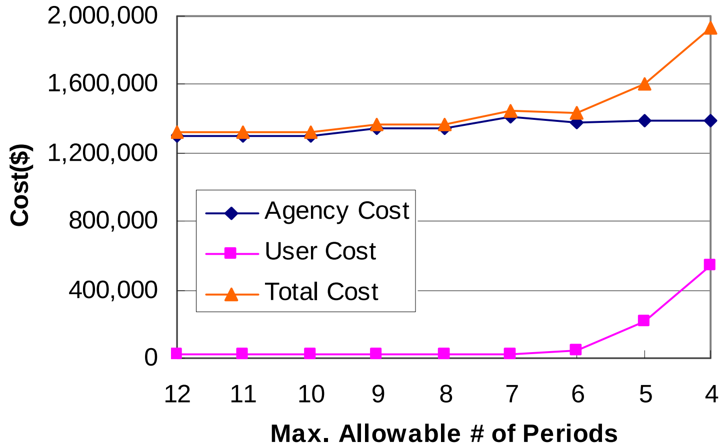
Figure 4-28 Work Zone Costs of Optimal Solutions with Varying Idling Cost

Table 4-22 Optimal Solutions with Varying Project Deadline (No Detour)

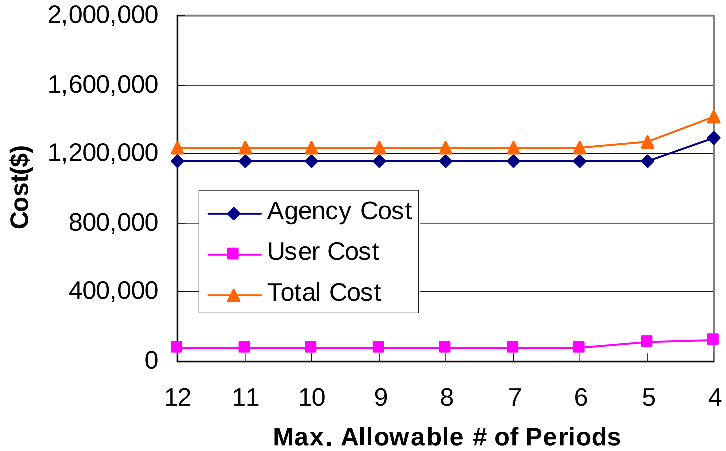
Max # of periods	***# of periods****	***Work Time***	***Zones***	***Str#1***	***Str#2***	***Str#3***	***Str#4***	***CA***	***CU***	***CT***
#	#	hr/ period	#	Lane Closure	Work Rate	Detour	Merge Control	$	$	$
12-10	10	10	20:30-06:30	double	normal	no	yes	1,295,360	22,719	1,318,080
9	8	10	20:30-06:30	double	medium	no	yes	1,341,890	23,078	1,364,970
8	8	10	20:30-06:30	double	medium	no	yes	1,341,890	23,078	1,364,970
7	7	10	20:30-06:30	double	fast	no	yes	1,417,950	23,335	1,441,290
6	6	11	20:00-07:00	double	fast	no	yes	1,381,060	49,132	1,430,190
5	5	15.5	19:30-07:00	double	fast	no	yes	1,384,270	217,517	1,601,790
			10:00-14:00	single	fast	no	yes			
4	4	17.5	19:00-07:30	double	fast	no	yes	1,385,430	541,280	1,926,710
			09:00-14:30	single	fast	no	yes			

Table 4-23 Optimal Solutions with Varying Project Deadline (Detour Control-SO)

Max # of periods	*# of periods**	*Work Time*	*Zones*	*Str#1*	*Str#2*	*Str#3*	*Str#4*	*CA*	*CU*	*CT*
#	#	hr/period	#	Lane Closure	Work Rate	Detour	Merge Control	$	$	$
12-6	6	17	19:30-07:00	double	normal	SO	yes	1,160,010	79,625	1,239,640
			09:00-14:00	double	normal	SO	yes			
5	5	18.5	19:00-08:00	double	normal	SO	yes	1,160,140	114,664	1,274,800
			09:00-14:30	double	medium	SO	yes			
4	4	19.5	19:00-08:00	double	medium	SO	yes	1,291,780	119,337	1,411,120
			08:00-14:30	single	medium	SO	yes			



(a) No Detour Control



(b) Detour Control-SO

Figure 4-29 Work Zone Costs of Optimal Solutions with Varying Project Deadline

4.6.4.4 Fixed Time

The fixed time for setting up a work zone may include mobilization time, pavement curing time, and demobilization time. In practice, fixed time usually increases with the intensity of the

maintenance. The impact of long fixed time is equivalent to the impact of low work rate. Table 4-24 shows the optimized solutions without detour control. As we can see, additional agency cost should be invested on accelerating the work in order to minimize the total work zone cost when fixed time exceeds 5 hr/zone. When fixed time is 8 hr/zone, the higher work efficiency gained from double-lane nighttime closure has to be sacrificed and single lane closure is adopted to buy longer work time. The optimization results with detour control (SO model) are provided in

Table 4-25. With increasing fixed time, the work zone scheduled in the daytime off-peak time window changes from 5-hr double lane closure to 6-hr single lane closure and it finally becomes unacceptable to set up any work zone in daytime off-peak hours because of extremely long fixed time (>=8 hr/zone). As shown in Figure 4-30, total work zone costs increases with the fixed time at a fast rate if no detour control is employed.

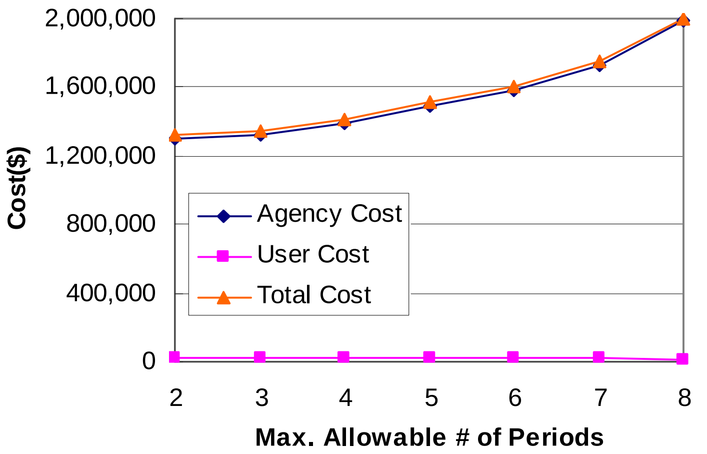
Table 4-24 Optimal Solutions with Varying Work Zone Fixed Time (No Detour)

z3	***# of periods****	***Work Time***	***Zones***	***Str#1***	***Str#2***	***Str#3***	***Str#4***	***CA***	***CU***	***CT***
hr/zone	#	hr/ period	#	Lane Closure	Work Rate	Detour	Merge Control	$	$	$
2	10	10	20:30-06:30	double	normal	no	yes	1,295,360	22,719	1,318,080
3	11	10	20:30-06:30	double	normal	no	yes	1,325,550	22,540	1,348,090
4	13	10	20:30-06:30	double	normal	no	yes	1,384,480	22,360	1,406,840
5	13	10	20:30-06:30	double	medium	no	yes	1,489,260	22,405	1,511,670
6	16	10	20:30-06:30	double	medium	no	yes	1,578,180	22,181	1,600,360
7	21	10	20:30-06:30	double	medium	no	yes	1,726,370	21,957	1,748,320
8	22	11.5	19:30-07:00	single	medium	no	yes	1,988,750	14,121	2,002,880

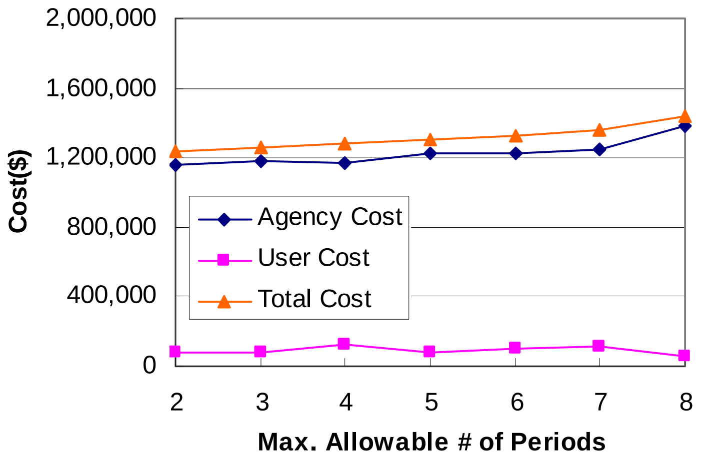
Table 4-25 Optimal Solutions with Varying Work Zone Fixed Time (No Detour Control-SO)

z3	***# of periods****	***Work Time***	***Zones***	***Str#1***	***Str#2***	***Str#3***	***Str#4***	***CA***	***CU***	***CT***
hr/zone	#	hr/ period	#	Lane Closure	Work Rate	Detour	Merge Control	$	$	$
2	6	17	19:00-07:00	double	normal	SO	yes	1,160,010	79,625	1,239,640
			09:00-14:00	double	normal	SO	yes			
3	7	17	19:00-07:00	double	normal	SO	yes	1,177,130	83,902	1,261,030
			09:00-14:00	double	normal	SO	yes			
4	7	19	19:00-08:00	double	normal	SO	yes	1,163,510	119,424	1,282,930
			09:00-15:00	double	normal	SO	yes			
5	9	19	19:00-08:00	double	normal	SO	yes	1,221,380	80,562	1,301,940

			09:00-15:00	single	normal	SO	yes			
6	10	19	19:00-08:00	double	normal	SO	yes	1,227,820	97,890	1,325,710
			09:00-15:00	single	normal	SO	yes			
7	12	21	19:00-08:00	double	normal	SO	yes	1,242,490	117,354	1,359,850
			08:00-16:00	single	normal	SO	yes			
8	14	13.5	19:00-8:30	double	normal	SO	yes	1,377,470	60,960	1,438,430



(a) No Detour Control



(b) Detour Control-SO

Figure 4-30 Work Zone Costs of Optimal Solutions with Varying Project Deadline

Although the Maryland State Highway Administration normally avoids scheduling maintenance activities on Friday and Saturday nights except in very special cases, weekend closure have been explored in other states such as California (*Lee et.al., 2006*) in large freeway rehabilitation projects where weekend traffic is significantly lower than weekday traffic and work intensity is high. To investigate the feasibility of weekend lane closure, an experiment is conducted to compare weekday work zone plan and weekend plan when fixed time is 8 hr/zone and detour control (SO) is employed. The optimization parameters for these two scenarios are listed in

Table 4-26 and traffic distributions are displayed in Figure 4-31. Note that idling time between two weekends are ignored in this test.

Table 4-27 provides the optimized weekday and weekend plans and Figure 4-32 shows the cost information associated with each plan. It can be seen with optimized weekend plan the maintenance work can be accomplished in less time and with lower total cost. It also causes less queuing delay, compared to weekday plan. Although no queue will form in weekend plan, the resulting user costs are higher than those resulted from weekday plan because more traffic are diverted to the alternative route which is longer and slower than the mainline route, therefore, the detour delay increases greatly.

This test demonstrates the potential benefit of weekend lane closures. However, many other factors have to be considered by transportation agencies during their decision making process. These include, for example, the acceptance and compliance of the travelers as well as the availability and quality of alternative routes.

Table 4-26 Optimization Parameters of Test Weekday and Weekend Plans

Optimization Parameters	Weekday Plan	Weekend Plan
The max # of periods	20 (480 hours)	2 (120 hours)
Starting time of a period	Weekday16:00	Friday 20:00
Duration of a period	24 hours	60 hours
Ending time of a period	16:00 next day	Monday 8:00

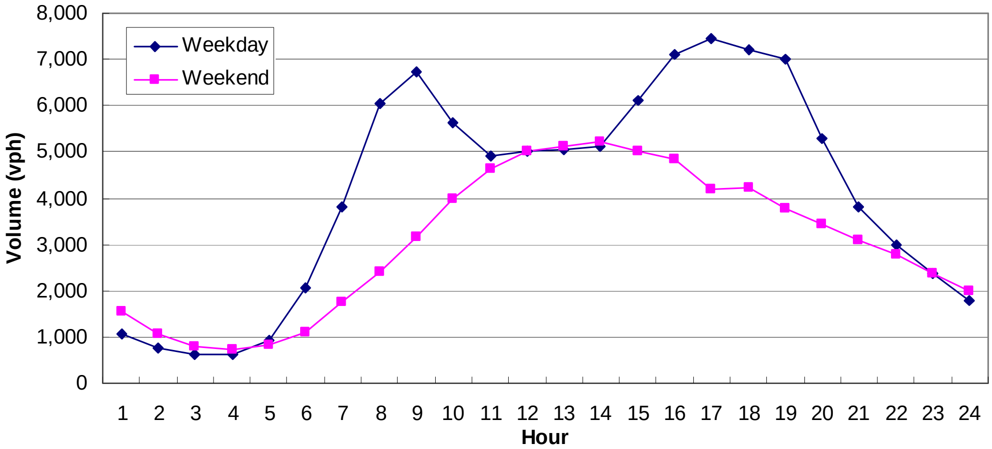


Figure 4-31 Weekday and Weekend Mainline Traffic Distributions

Table 4-27 Optimized Weekday and Weekend Plans

z3	***# of periods****	***Work Time***	***Zones***	***Str#1***	***Str#2***	***Str#3***	***Str#4***	***Work Zone Total Cost***	***Max. Queue Length***	***Total Closure Time***
hr/zone	#	hr/ period	#	Lane Closure	Work Rate	Detour	Merge Control	$	mile	hour
Weekday Plan										
8	14	13.5	19:00-8:30	double	normal	SO	yes	**1,438,430**	**1.45**	**189**
Weekend Plan										
8	2	55.5	Fri. 20:00-Sat. 16:00	double	normal	SO	yes	**1,370,870**	**0**	**111**
			Sat. 19:30-Mon. 07:00	double	normal	SO	yes			

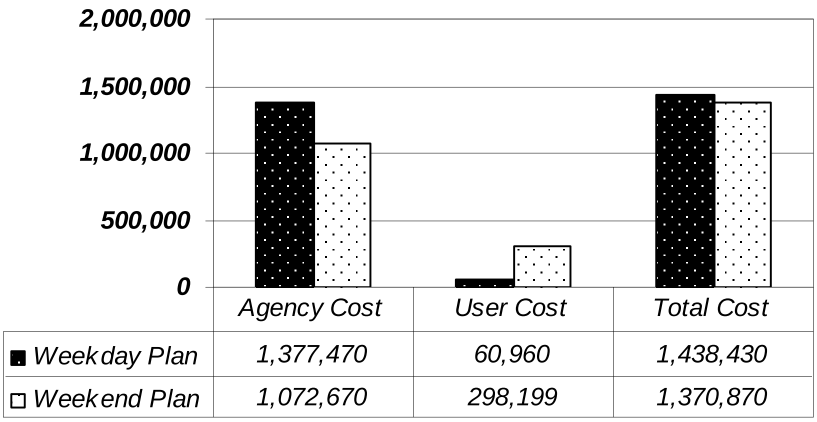


Figure 4-32 Work Zone Costs of Optimized Weekday and Weekend Plans

4.6.5 Findings

The numerical experiment tests the performance of the proposed work zone decision optimization method. The convergence, optimality and reliability analysis for four test

scenarios demonstrate that the proposed optimization algorithm is reliable to obtain near-optimal solutions. Since a low traffic level may enlarge the feasible solution space, increasing the population size at the first stage and the number of generations at the second stage is recommended to ensure the quality of the solution. Sensitivity analysis is conducted to investigate the impact of traffic level, detour fraction estimation model, resource idling cost, project deadline, and fixed time on the optimized work zone management plan. We draw the following conclusions based on the analysis of optimization results:

(1) Efficient lane closure tactics (e.g. scheduling work zone in appropriate time windows) can significantly reduce the work zone costs.

(2) Deployment of traffic impact management strategies, such as merge control and detour control system, can be beneficial and cost effective, especially on projects with high resource/labor idling cost, longer fixed work zone setup time, and tighter deadline.

(3) Detour control has great potential to mitigate the traffic impact and reduce project cost by efficiently utilizing spare capacity in a road network. Its effectiveness depends highly on road users' detour behavior as well as the physical and traffic characteristics of the mainline and alternative routes.

Chapter 5 Short-term Work Zone Decision Optimization Based on Simulation

The accuracy of user delay estimates significantly affects the measure of the total work zone cost. Microscopic simulation programs, which model each vehicle as a separate entity, are usually expected to provide more accurate estimates of vehicle speeds and delays compared to analytical procedures, especially when the traffic conditions or roadway networks are complex. Therefore, a work zone optimization model based on simulation is introduced in this chapter. With the same optimization algorithm 2PBSA, this simulation method is applied instead of the analytic method to evaluate the objective function in the optimization process.

It should be noted that such optimization through microscopic simulation is computationally intensive. To make the search algorithm as efficient as possible and thus reduce the computational burdens to a more acceptable level, a hybrid approach combining simulation and analytic methods is also proposed in this chapter. In addition, a parallel computing technique is applied to further reduce the computation time.

5.1 Problem Statement

5.1.1 Pros and Cons of Simulation Method

Microscopic simulation models, such as CORSIM, are powerful operational-level traffic analysis tools to analyze key bottlenecks on roadway segments and corridors where the movement of each individual vehicle needs to be represented to better understand the impact on roadway conditions (*FHWA, 2008*). They are extensively used to access detailed system-level work zone impacts and evaluate potential work

zone management strategies especially in significant projects which may have great effects on traffic conditions in or around work zones or at locations with complex geometric configurations. Although the analytical traffic impact assessment model developed in Chapter 3 is based on simulation results, it may still be out-performed by microscopic simulation models which require less overly-simplified assumptions on network configuration, drivers' response to different traffic management strategies and other important traffic features.

However, the following limitations of the simulation models often preclude them from being adopted by agencies:

- Substantial amount of roadway geometry, traffic control, and traffic pattern data is required to collect and code in the simulation model.
- It takes extensive time and resource to calibrate a simulation model so that it matches the actual conditions.
- Heavy computational burden is associated with simulation especially at a microscopic level. Due to the stochastic nature of simulation, multiple simulation replications are needed obtain a statistically significant estimate of the performance of a particular design, which further increase the simulation time.
- Other restrictions in simulation models may bias the results. Such issues may be specified to particular models. For example, the experiment conducted in Chapter 3 shows that when work zone impact is evaluated by CORSIM, the

running time and variance of simulation results increase with traffic congestion level; when the congestion level rises beyond acceptable bounds, the simulation results become unreliable.

5.1.2 Problem Formulation

The study presented in this chapter focuses on solving short-term decision optimization problem for intermediate recurrent work zones on freeways. In chapter 3, an optimization model (Model 1-2) has been developed for recurrent work zones and we adopt the model in this chapter.

Model 1-2

Objective:

Min $C_T(m, \vec{X}) = C'_T(m', \vec{X}) \dfrac{L_T}{L'_T}$

Subject to:

$$T_s \leqslant E_{i-1} \leqslant S_i \leqslant E_i \leqslant T_e \tag{1}$$

$$(E_{m'} - S_1) \leqslant D'_T \tag{2}$$

$$(E_i - S_i) > z_3 \tag{3}$$

$$L_T \frac{D'_T}{D_T} \le L'_T \le L_T \qquad \text{where } L'_T = \sum_{i=1}^{m'} L_{wi} N_{wi} \tag{4}$$

$$\max\{q(t)\} \le q_{\max} \tag{5}$$

where,

C_T = Total work zone cost of the project;
C'_T = Total work zone cost in one cyclic period;
L_T = Total lane-mile to be maintained in the project;
L'_T = Total lane-mile maintained in one cyclic period;
m = Total number of work zones needed to complete the project;
m' = Total number of work zones set up in one cyclic period;
$\vec{X}$ = Short-term work zone decisions;
T_s = Starting time of a cyclic period;
T_e = Ending time of a cyclic period;
D'_T = The duration of a cyclic period, $D'_T = T_e - T_s$;
S_i = Starting time of the i^{th} work zone;
E_i = Ending time of the i^{th} work zone;
L_{wi} = Length of the i^{th} work zone;

N_{wi} = Number of closed lanes in the i^{th} work zone;
$q(t)$ = Queue length at time t;
q_m = Maximum acceptable queue;
$C_{E,i}$ = User Expected Accident Cost of the i^{th} work zone;
z_1 = The fixed setup cost per work zone;
z_3 = The fixed setup time per work zone;
z_2 = The unit length maintenance cost;
z_4 = The unit length maintenance time.

5.1.3 Optimization Algorithm

The two-stage population-based simulated annealing (2PBSA) proposed in Chapter 4 is applied to solve the work zone optimization problem. The optimization method includes four major elements: (1) Initial solution generation; (2) New solution generation; (3) Solution evaluation and (4) Solution search algorithm. In Chapter 4, the third part, solution evaluation, is based on an analytical method while in this chapter, a simulation model is employed to estimate work zone user delays. The basic procedure of the PBSA and the two-stage feature are illustrated in Figure 5-1 and Figure 5-2.

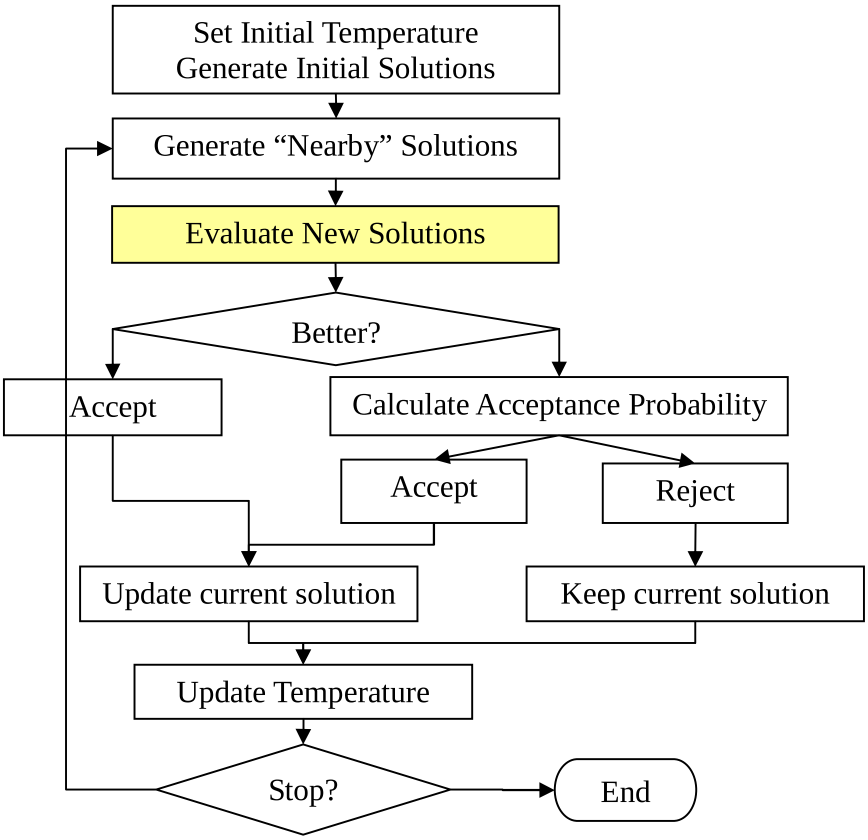


Figure 5-1 Basic Procedure of PBSA

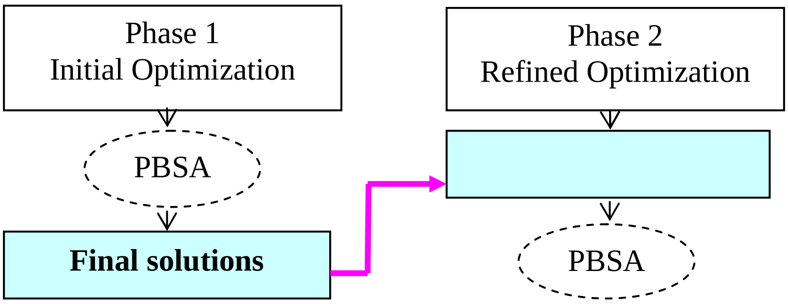


Figure 5-2 Concept of 2-Stage PBSA

5.2 Solution Evaluation Based on Simulation

In this study, simulation based on CORSIM is performed to evaluate the user delays (t_d) caused by the work zone activity. The procedure for simulating different work zone characteristics has been introduced in Chapter 3. However, to generate simulation input files and obtain simulation output in an automated way, three modules are needed to

link the optimization process with the CORSIM model, shown in Figure 5-3. These three modules are the preparation module, the preprocessor module and the postprocessor module.

1. Required Inputs

Before evaluating solutions, users must provide the following inputs:

(1) The well-calibrated simulation parameters, including rubbernecking factor, car following multiplier, link free flow speed, etc., associated with the work zone characteristics to be optimized;

(2) Two CORSIM input files with the format of *.trf file, which provide datasets describing geometrics of the study network, 24-hour traffic information and traffic control parameters. Each input file includes 12 time periods with 1 hour for each time period. Hourly time-varying traffic information from 0:00 to 12:00 is recorded in the first CORSIM input file (Morning 12-hour Simulation Input File). Hourly time-varying traffic information from 12:00 to 24:00 is recorded in the second CORSIM input file (Afternoon12-hour Simulation Input File).

2. Preparation Module

The Preparation Module is used to provide some of required data needed in the preprocessor and postprocessor modules. The framework of the preparation module is displayed in Figure 5-4.

Step1: For the Morning 12-hour Simulation Input File and the Afternoon 12-hour Simulation Input File, call CORSIM.DLL to run simulation. Two output files can be obtained after the simulation is completed.

Step 2: From the output files, get hourly traffic volumes in each link and hourly network-wide delay time. The former will be used to get peak hours in the solution generation process and to calculate new turn movement percentages with detours in the preprocessor module. The latter will be used in the postprocessor module to calculate user delays in a normal situation without a work zone.

3. Preprocessor Module

The purpose of the Preprocessor Module is to generate new CORSIM input files according to the work zone information in the candidate solution generated from the optimization process. Figure 5-5 shows the flow chart of the preprocessor module.

Step 1: According to the work zone characteristics provided by the solution, calculate the total time period need to simulate.

Step 2: Due to the limitation of no more than 19 time periods in CORSIM, more than one input file may have to be generated for simulating the work zone activity. Based on the Morning 12-hour Simulation Input File and the Afternoon 12-hour Simulation Input File, generate the input files with different simulation start time and periods. Note that in these input files no work zone information is recorded.

Step 3: According to the work zone information in the solution, modify the input files generated in step 2. The details of the modification procedure have been introduced in

Chapter 3. After the modifications, new CORSIM input files with work zone information can be obtained.

Step 4: For these new CORSIM input files, call CORSIM.DLL to run simulation.

4. Postprocessor Module

The objective of the Postprocessor Module is to interpret the CORSIM outputs to the objective function values, which should be send back to the optimization process. The steps are demonstrated in Figure 5-6.

Step 1: Read the network-wide delay times from the simulation outputs of the CORSIM input files generated in the preprocessor module.

Step 3: Calculate the user delay in work zone conditions.

Step 4: Calculate the user delay in a normal situation without work zones according to the and hourly network-wide delay time obtained in the preparation module.

Step 5: Calculate the used delay caused by work zone activity defined in the candidate solution by subtracting the delay without work zones from the delay with work zones.

Step 6: Calculate the user delay costs based on the user delay obtained from the simulation model. The other cost components are still calculated in an analytical way.

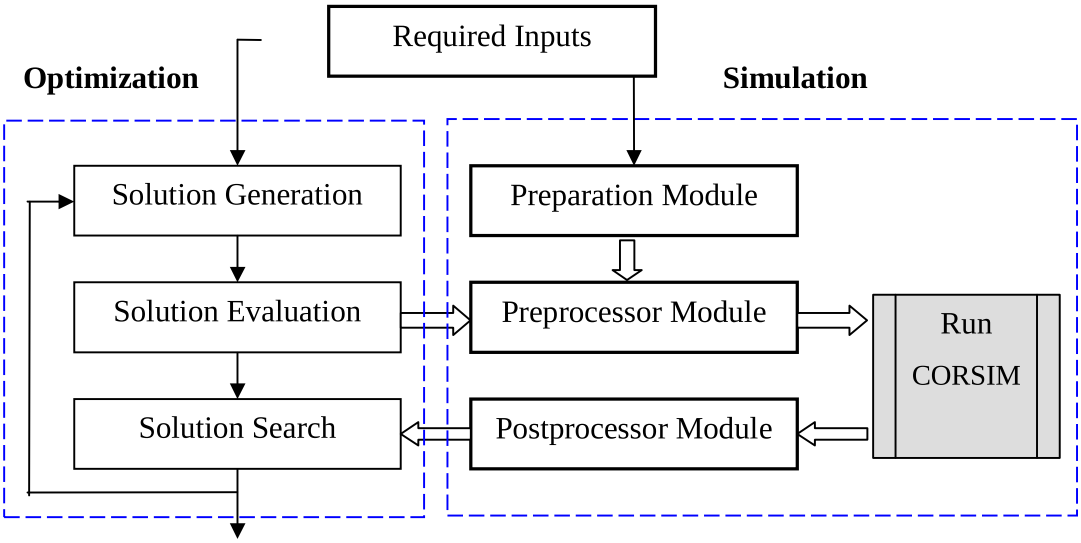


Figure 5-3 Links between Optimization Process and Simulation Process

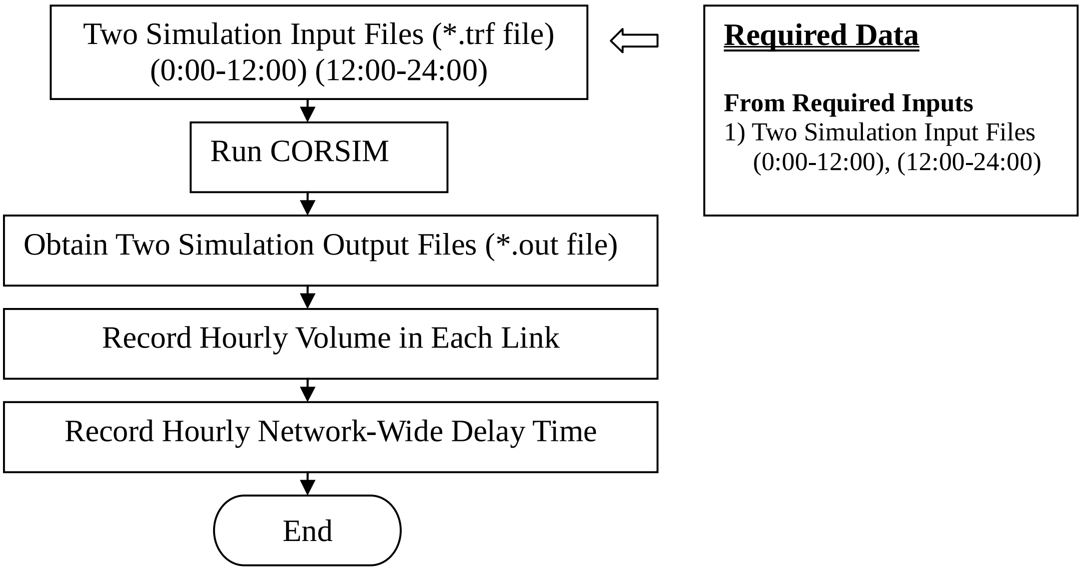


Figure 5-4 Framework of the Preparation Module

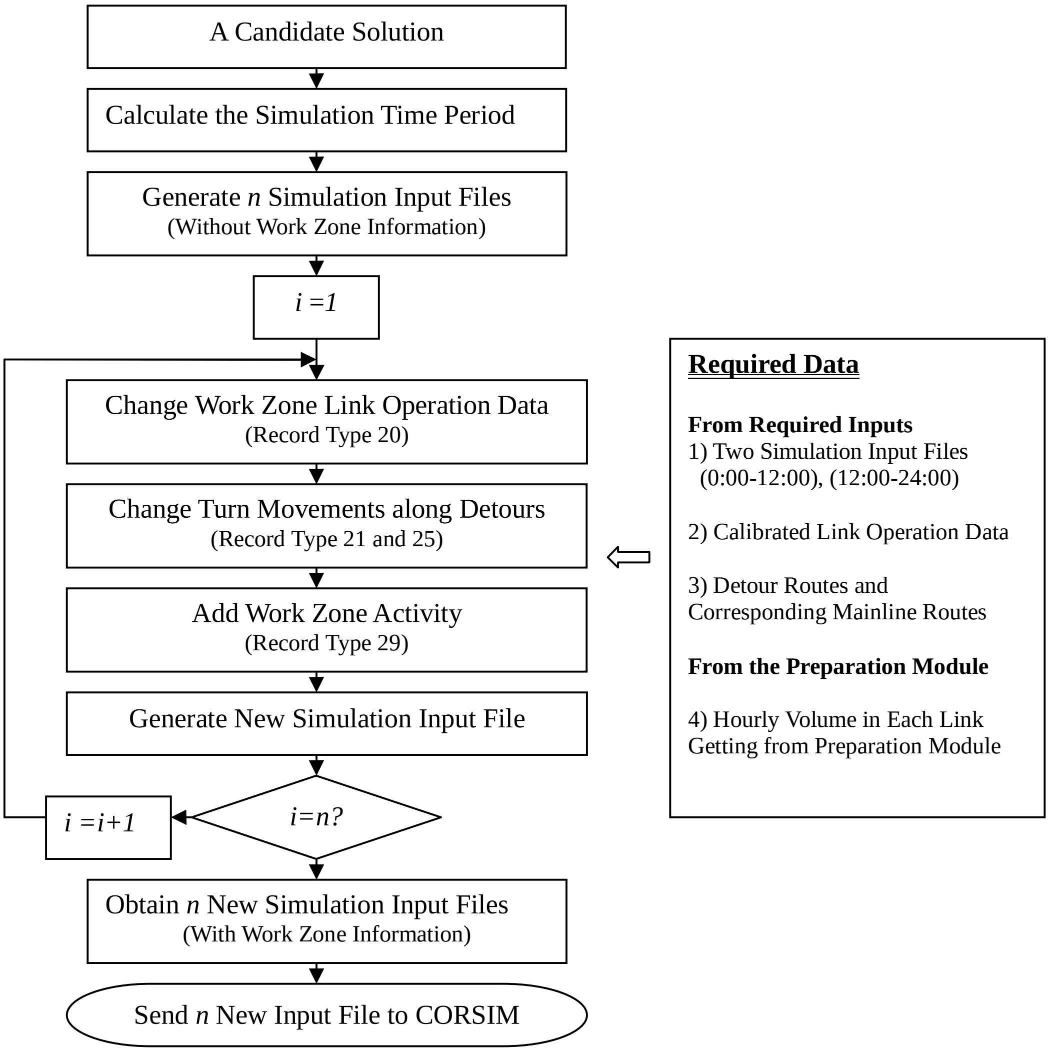


Figure 5-5 Framework of the Preprocessor Module

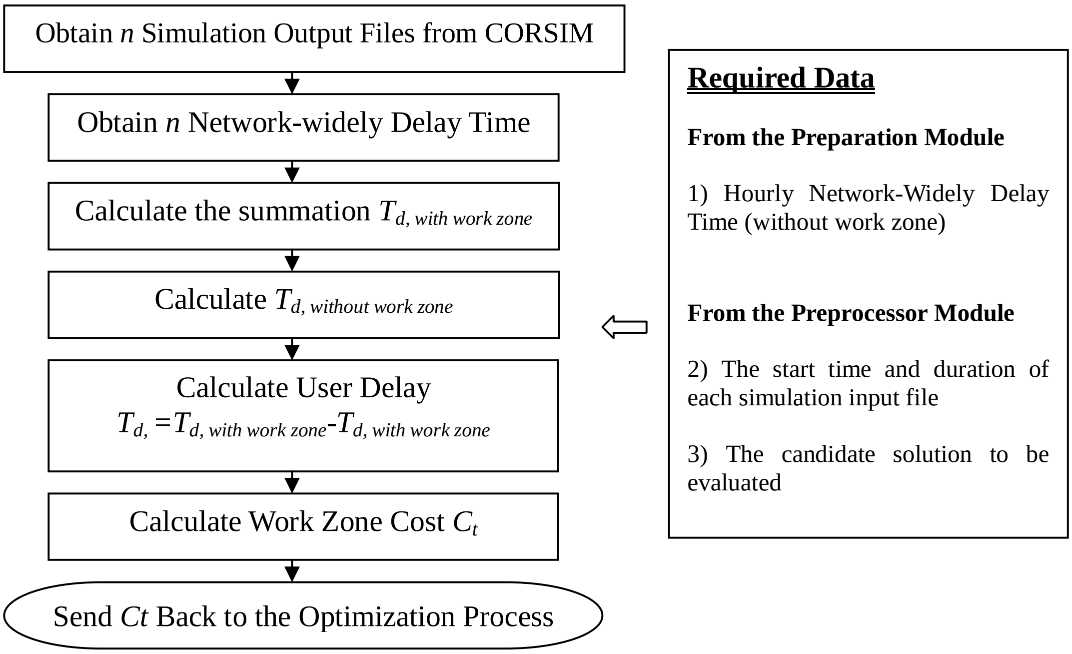


Figure 5-6 Framework of the Postprocessor Module

5.3 Methods for speeding up Simulation-based Optimization

In order to reduce the computational burden while still estimating precise work zone costs, two methods are proposed in this subsection. One is to improve the solution search efficiency by pre-optimizing the decision variables based on an analytical model before performing simulation-based optimization. The other is to incorporate parallel computing into the simulation-based optimization for accelerating the solution evacuation process.

5.3.1 Hybrid Method

In this method, the analytical method derived from simulation results is applied to evaluate the objective function in the first stage of the two-stage PBSA algorithm. Initial optimization based on the analytic model is performed and the result will be sent to the second stage as a relatively good initial solution. In the second stage, the

optimization model based on simulation method performs a refined search inside the promising region provided by the first stage.

Through this hybrid approach, complete simulations can be avoided in the early search phases. The optimizing search process based on simulation method can start from pre-optimized decision variables and thus may be able to reach a high quality optimized solution in an efficient way. The hybrid method will thus combine the benefits of the wide search algorithm and the local search algorithm as well as integrate the advantages of macroscopic analytic methods and microscopic simulation methods.

Before optimizing through the hybrid method, it is important to calibrate well the simulation model to reflect the actual situation in the real-world and also calibrate the input parameters, such as capacity, average speed in analytical model to maintain the consistency between the analytic model and simulation models. Note that the study network should be simplified into the networks explored with the analytic models.

5.3.2 Application of Parallel Computing

Parallel computing is the simultaneous use of multiple computer resources to solve a problem in order to obtain results faster. The idea is based on the fact that the process of solving a problem can be divided into smaller tasks, which may be carried out simultaneously with some coordination. There have been also a wide range of applications incorporating parallelism into optimization methods, such as parallel branch-and-bound algorithms (*Gendron and Crainic, 1994*) and parallel metaheuristics (*Alba et.al., 2005*). For NP-hard combinatorial optimization problems, metaheuristics are more frequently used than exact search algorithms. Among those

metaheuristics, population-based algorithms are particularly easy to implement and promises substantial gains in performance because the procedure of evaluating multiple solutions is naturally prone to parallelism. For speeding up the optimization process, the 2PBSA algorithm is re-programmed for parallel computing. Figure 1 shows the PBSA procedure with the master-slave model.

(1) Parallelization model

The major concept of the parallel 2PBSA algorithm is to distribute the computations of the objective functions over multiple processors, computers, or workstation networks. We implement this concept using a master-slave parallelization paradigm (*Cantu-Paz, 1997*), in which a "master processor" synchronizes and controls the main loop of the solution search procedure while multiple "slave processors" executes solution evaluation tasks.

At each solution evacuation step, the "master processor" distributes computational tasks and necessary parameters necessary to the "slave processors". The "slave processors" receive the messages, complete the tasks, and then return the results to the "master processor". Figure 5-7 shows the PBSA procedure with the master-slave model. The parallel programming is based on Message-Passing Interface (MPI) (*Pacheco, 1998; Paul, 2005*), which is a widely-used library of functions and macros that can be used in C, FORTRAN, and C++ programs.

(2) Task Distribution

From a direct point of view, it is natural to define “evaluating the objective function value of an individual” as a “task” which is assigned to a processor. The parallelized solution evacuation procedure consists of three phases, as illustrated in Figure 5-8. In the first phase, the master process divides all individuals into subgroups as equally as possible according to the number of individuals along with the number of available processors and sends each subgroup to its corresponding processor. In the second phase, each processor receives a group of individuals. It runs several simulation replications for each individual, and then returns the average results to the master processor. In the third phase, the master processor collects all evaluation results after completing its own evaluation task, and then completes the whole procedure. With this preset task distribution method, all available processors including the master processor can contribute to the solution evaluations.

Note that there is another possible way to distribute tasks. Instead of distributing individuals (solutions), a simulation replication with a preset random seed associated with an individual is treated as a unit of task. After evaluation results of all replications have been returned, the master processor takes the average and matches the average value to each individual in the current population. The potential benefit of the approach is to reduce the load imbalance caused by variance of simulation time of different individuals, which may lead to different network congestion level. In the current study, this approach is not applied. This may be worth exploring in the future studies.

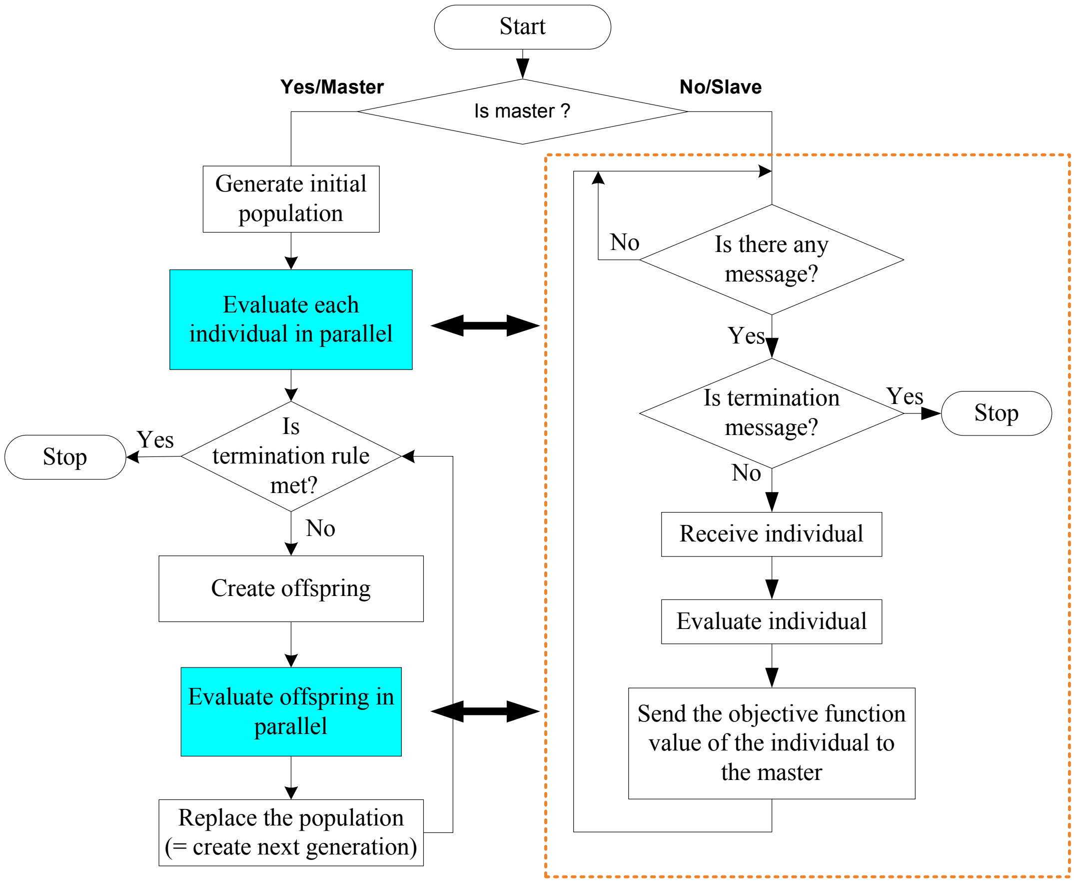


Figure 5-7 PBSA Procedure with the Master-Slave Model

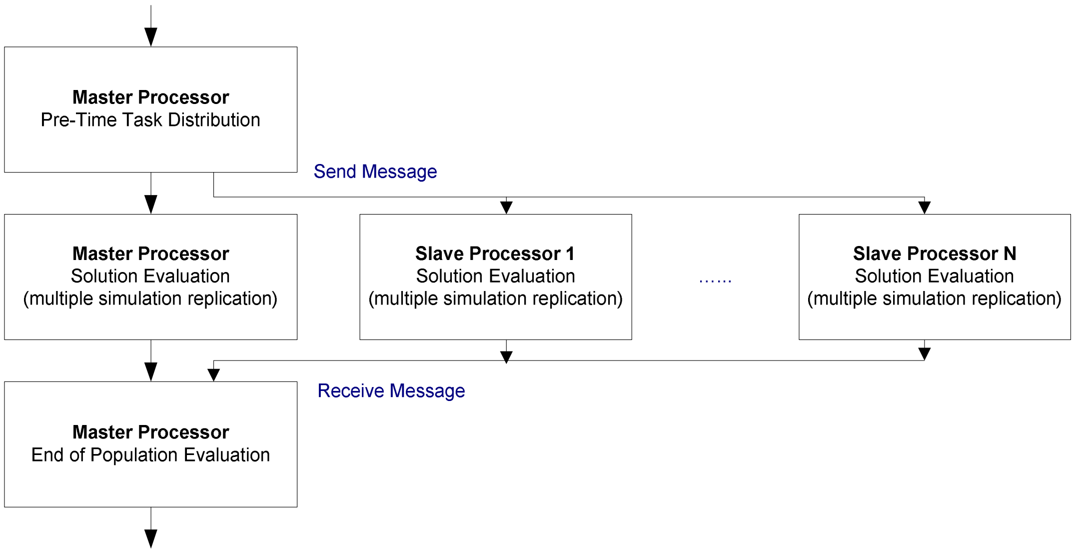


Figure 5-8 Parallelized Solution Evacuation Procedure

5.4 Numerical Experiment

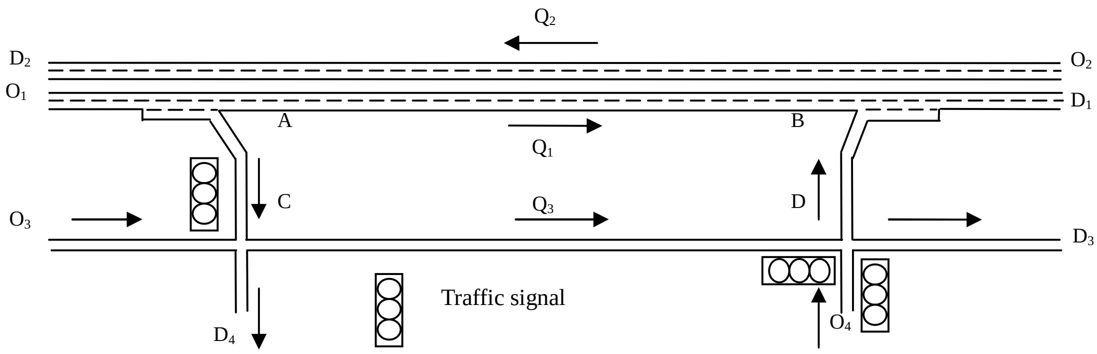
5.4.1 Test Network

A hypothetical network with multiple origins and destinations, shown in Figure 5-9 (a), is conceived in order to demonstrate the methodologies presented in this chapter. The network consists of a corridor with a four-lane two-way freeway and a parallel arterial. The freeway is 3.11 miles long. Both off-ramp deceleration lanes and on-ramp acceleration lanes are 800 feet long. The single-lane arterial is unidirectional. An actuated signal alternates permission between the off-ramp and the arterial. The arterial approaches to the on-ramp are controlled by a pre-timed signal control. The network is coded in CORSIM (Figure 5-9 (b)) and it is simplified to an analytic model (Figure 5-9 (c)). Both lanes on the freeway section AB are to be maintained and therefore in total 6.22 lane-miles need to be maintained. It is assumed that lane closures are prohibited in morning peak hours from 6:00 am to 10:00 am, i.e., the work zone activity is restricted from 10: 00 am to 6:00 am the next day. Therefore, our analysis is based on a 20-hour cyclic period from 10:00 am to 6:00 am the next day.

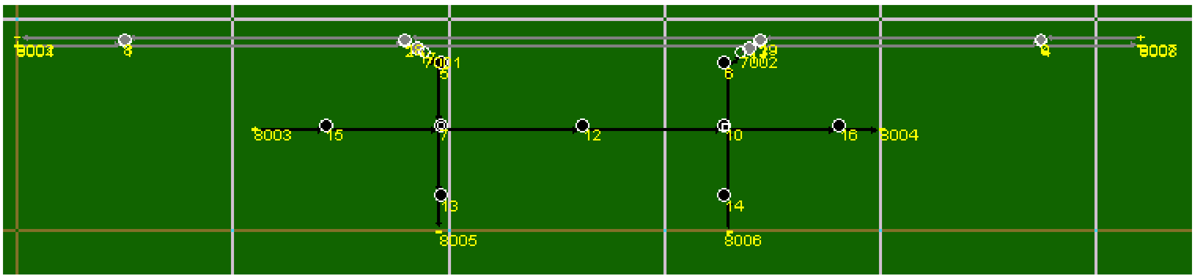
The baseline numerical values are shown in Table 5-1. Three work rates are available for selection, as shown in Table 5-2.

Table 5-3Table 5-3 provides the assumed baseline traffic distributions on the mainline and detour over each day. To calibrate the simulation model, two simulation parameters, the car following sensitivity factor and rubbernecking factor, are tuned to equalize the maximum hourly throughputs to theoretical roadway capacities. Since CORSIM MOE related to vehicle operating cost cannot provide reliable estimate (*FHWA, 2007)*, user vehicle operating cost is not considered in this study. Idling cost is also set to 0 in this test.

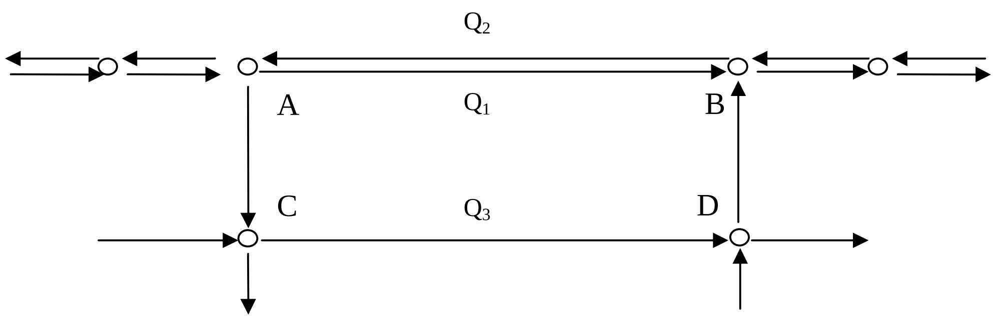
All experiments are run on four PCs. Each PC has 0.99 GB of RAM and two Pentium ® IV3.60 GHz processors. Since using two processors which shares memory in the same computer will considerably increase the simulation time in each processor due to memory access conflict, only one processor is used in each computer.



(a) The Study Road Network



(b) The Simulation Model of the Study Road Network in CORSIM



(c) The Analytic Model of the Study Road Network

Figure 5-9 The Analytic Model and Simulation Model of the Study Network

Table 5-1 Notation and Baseline Numerical Inputs

Variable	Description	Value
L_{AB}	Length of Segment AB	3.11 miles
L_{AC}	Length of Segment AC	0.93 miles
L_{CD}	Length of Segment CD	2.49 miles
L_{DB}	Length of Segment DB	0.93 miles
N_{AB}	Number of lanes in Segment AB	2 lanes
N_{CD}	Number of lanes in Segment CD	1 lane
c_0	Maximum discharge rate without work zone	2,200 vph /lane
c_w	Maximum discharge rate with work zone	1,600 vph /lane
V_{AB}	Average approaching speed	65 mph
Vw	Average work zone speed	55 mph
V_{CD}	Free flow speed in Segment CD	45 mph
$V_{AC/DB}$	Average speed in Segment AC/DB	45 mph
T_{int}	Average waiting time passing intersections along the detour	30 seconds/veh
n_a	Number of crashes per 100 million vehicle hours	40 acc/100mvh
v_a	Average accident cost	142,000$/accident
v_d	Value of user time	12 $/veh·hr
N_w	Number of closed lanes in Direction 1	1 lane
N_c	Number of usable counter flow lanes in Direction 2	0 lane
N_a	Number of access lanes	0 lane
P_{max}	Allowable maximum diverted fraction	30%

Table 5-2 Candidate Work Rates

Work Rate	z_1 ($/zone)	z_2 ($/lane.mile)	z_3 (hr/zone)	z_4 (hr/lane.mile)
Rate A	1,000	32,000	2	12
Rate B (baseline)	1,000	33,000	2	10
Rate C	1,000	34,000	2	8

Table 5-3 AADT and Hourly Traffic Distribution in the Study Network

Time Period	Time	Mainline		Detour
		Q_1 (vph)	Q_2 (vph)	Q_3 (vph)
0	0:00-1:00	220	930	392
1	1:00-2:00	157	645	391
2	2:00-3:00	148	301	367
3	3:00-4:00	198	238	432
4	4:00-5:00	448	240	432
5	5:00-6:00	1,425	326	432
6	6:00-7:00	2,941	580	734
7	7:00-8:00	3,541	887	1,276
8	8:00-9:00	2,897	977	1,505
9	9:00-10:00	2,509	1,134	1,363
10	10:00-11:00	1,793	1,283	951
11	11:00-12:00	1,586	1,589	772
12	12:00-13:00	1,528	1,544	700
13	13:00-14:00	1,475	1,673	670
14	14:00-15:00	1,541	2,074	773
15	15:00-16:00	1,414	2,808	954
16	16:00-17:00	1,079	3,501	1,042
17	17:00-18:00	957	3,719	1,026
18	18:00-19:00	991	3,061	832
19	19:00-20:00	779	2,171	770
20	20:00-21:00	554	1,433	644
21	21:00-22:00	504	1,314	559
22	22:00-23:00	436	905	392
23	23:00-24:00	325	720	391
AADT		29,446	34,053	17,800
Average	Hourly	1227	1419	742

5.4.2 Optimization Results

For comparison, two work zone optimization models, with and without applying hybrid method, are applied separately to search for the optimal work zone plans for the example problem using the parallel two-stage Population-Based Simulated Annealing algorithm (2PBSA) with the parameters listed in Table 5-4. The two cases are denoted as S-2PBSA and H-PBSA, respectively.

Table 5-4 Algorithm Parameters in 2PBSA with and without using Hybrid Method

Stage	PBSA Algorithm Parameter	S-2PBSA	H-2PBSA
1	Solution Evaluation Method	Simulation	Analytical Model
	# of Generations	11	11
	Population Size	10	10
	Simulation Replication	5	N/A
2	Solution Evaluation Method	Simulation	Simulation
	# of Generations	11	11
	Population Size	2	2
	Simulation Replication	5	5

Table 5-5 provides the optimized results. H-2PBSA yields the same optimized solution as S-2PBSA, with a 0.9 mile length zone, work rate A and 13-hour time window from 4:00 pm to 5:00 am next day. Compared to two current policies used by the Maryland State Highway Administration (MDSHA), the optimized results yield lower total cost per lane mile, as shown in Table 5-6.

Table 5-5 Optimized Results of S-2PBSA and H-2PBSA

Optimized Results	S-2PBSA	H-2PBSA
Time Window	16:00-5:00	(16:00-5:00)
Work Duration	13 hours	13 hours
Work Zone Length	0.9 miles	0.9 miles
# of Closed Lane	1	1
Work Rate	Rate A	Rate A
# of Periods Needed	7	7
Agency Cost ($/lane.mile)	33,090	33,090
User Delay Cost ($/lane.mile)	147	147
Total Cost ($/lane.mile)	33,238	33,238

Table 5-6 Comparison of the Optimized Results and Current Policies

Optimized Results	S-2PBSA	Current Policy 1	Current Policy 2
Time Window	16:00-5:00	9:00-15:00	17:00-5:00
Work Duration	13 hours	6 hours	12 hours
Work Zone Length	0.9 miles	0.33 mile	0.83 mile
# of Closed Lane	1	1	1
Work Rate	Rate A	Rate B	Rate B
# of Periods Needed	7	15	7
Agency Cost ($/lane.mile)	33,090	35,500	34,000
User Delay Cost ($/lane.mile)	147	208,817	98
Total Cost ($/lane.mile)	33,238	244,371	34,098

Figure 5-10 shows how the optimization processes converge. It can be seen that in the first stage the PBSA algorithm finds relatively good solutions in both cases. In H-2PBSA case, the PBSA at second stage works quite efficiently to search for better solutions through simulation within the relatively good neighborhoods provided by the first stage.

Table 5-7 and Figure 5-11 show the performances of the parallel 2-PBSA when using different numbers of processors. As expected, the running time decreases as we add processors. H-2PBSA uses much less running time than S-2PBSA because fewer solutions are evaluated through simulation.

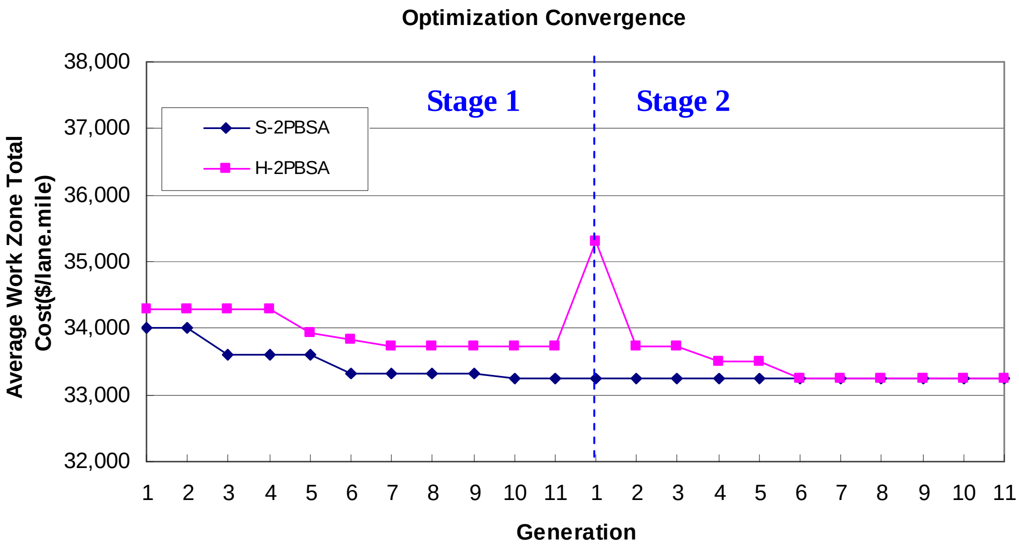


Figure 5-10 Convergence toward Optimality of S-2PBSA and H-2PBSA

Table 5-7 Running Time with Varying Number of Processors

Running Time	S-2PBSA	H-2PBSA
1 Processor	31.8 hours	6.3 hours
2 Processors	17.4 hours	3.4 hours
3 Processors	14.0 hours	2.5 hours
4 Processors	11.6 hours	1.9 hours

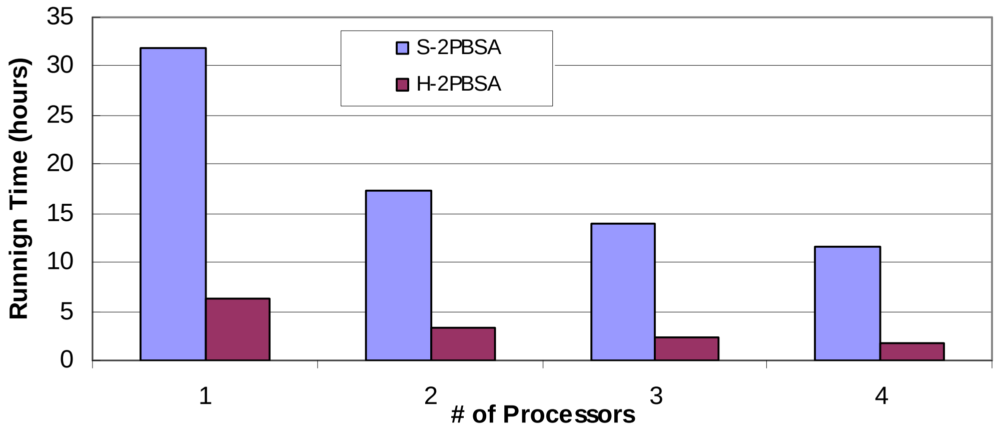


Figure 5-11 Running Time with Varying Number of Processors

5.4.3 Findings

In this experiment, the methodology for optimizing short-term work zone decisions based on simulation is tested. Two methods, hybrid method and parallel computing, proposed to reduce the computational burden imposed by the simulation process are examined. From the results, we obtain the following findings:

- The simulation-based work zone decision optimization without using any speed-up methods is quite time-consuming, even for a relatively small problem.

- The hybrid method combines the advantages of the analytical method (quickness) and simulation (more precision). It can yield satisfactory solutions, which are close to simulation-based optimization results, but obtained with much less computation time.

- Experimentation demonstrates the effectiveness of the parallel computing techniques.

Chapter 6 Joint Optimization of Short-term and Long-term Decisions

When competing alternatives resulting in different future pavement performance are taken into account in a rehabilitation project, one-time work zone cost is no longer suitable for measuring their cost-effectiveness. Instead, a customized cost-effectiveness index, which accounts for both pavement performance and life-cycle cost, may be employed as measure of effectiveness. This chapter starts by modeling how a long-term pavement decision affects the life-cycle agency cost and user cost. After that, a methodology jointly optimizing short-term work zone decisions and long-term pavement decisions is developed to maximize the cost effectiveness of the maintenance activity.

6.1 Problem Statement

The selection of paving strategies, including different combinations of layer material and layer thickness, is an important decision in highway rehabilitation projects. It is desirable to construct a pavement section that provides a long life and thus reduce future agency and user costs due to higher overall level of serviceability and less frequent maintenance and rehabilitation activities. However, longer-lived pavements may increase the one-time work zone agency and user costs due to more expensive construction cost and longer lane closures. Figure 6-1 depicts the difference in performance levels and corresponding costs for different rehabilitation strategies. Therefore, a tradeoff between short-tem costs and long-term costs has to be made when determining pavement design features and traffic management infrastructure in rehabilitation projects.

A reasonable approach for evaluating maintenance decisions is Life Cycle Cost Analysis (LCCA), which combines all costs and all recurring future expenditures into Net Present Value (NPV) or Equivalent Uniform Annual Costs (EUAC) over the analysis period. NPV and EUAC are calculated with the following equations:

$$NPV = C_T + \sum_{t=0}^{n} C_{LT}(t)\left[\frac{1}{(1+i)^t}\right] \qquad \text{Eq.6-1}$$

$$EUAC = NPV\,\frac{i(1+i)^n}{(1+i)^n - 1} \qquad \text{Eq.6-2}$$

where, C_T = one-time work zone cost;
$C_{LT,}(t)$= future cost in year t;
i = interest rate;
n = the number of years of the analysis period;

Involving the long-term user operating costs or safety costs and formulating them as functions of pavement conditions is a way to consider the effect on pavement condition in LCCA analysis. However, long-term user costs are usually difficult to quantify in monetary terms and even if they are quantified, they tend to overwhelm agency costs, particularly in high-volume roadways, which may mask significant agency cost differences among alternatives. Therefore, LCCA is appropriately applied only to compare project implementation alternatives that would result in the same level of service and benefits to the project. In order to evacuate and comparing competing pavement strategies that may yield dissimilar pavement performance levels, a cost effectiveness index (CEI) involving the use of the pavement performance curve, the traffic growth curve, and the Life Cycle Cost, is proposed as the economic indicator for assessing and quantifying the nonmonetary and monetary benefits and costs. The preferred paving strategy is the one that maximizes this CEI.

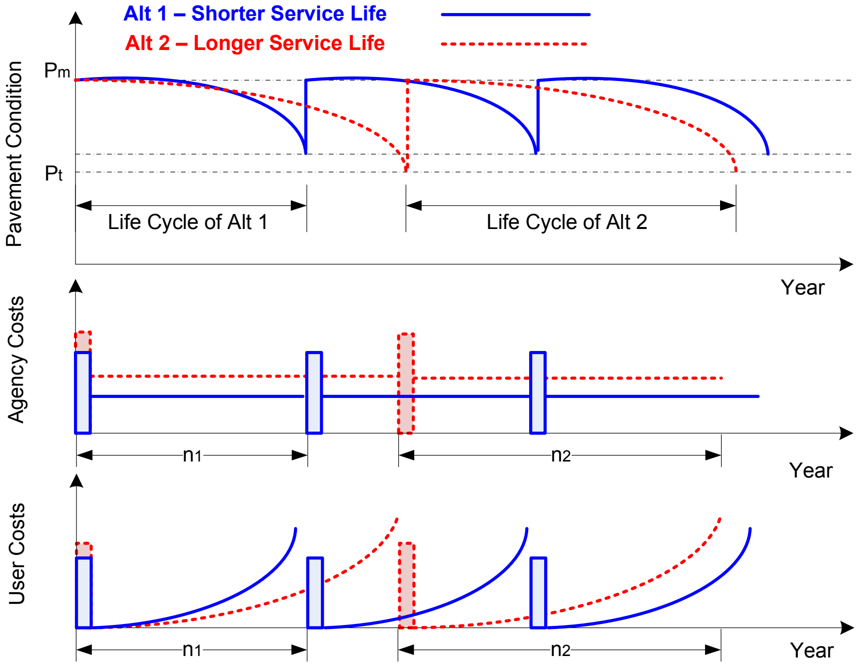


Figure 6-1 Pavement Strategies and the Corresponding Performance and Costs

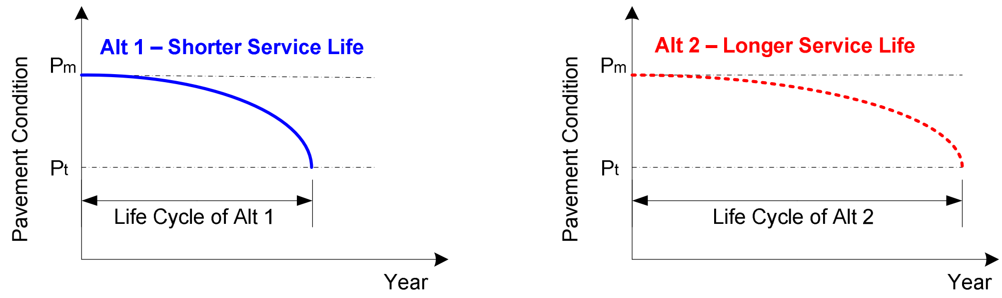
6.2 Study Scope and Assumptions

This study focuses on optimizing pavement decisions in highway rehabilitation activities, and specifically in pavement resurfacing (overlay) projects.

Assuming that the pavement deterioration and improvement models are deterministic and follow the Markov property[3] over a long planning horizon, a steady state can be reached by conducting the same measure of rehabilitation whenever pavement deteriorates from the restored serviceability level (P_m) to a threshold serviceability level (P_t) (*Tsunokawa and Schofer, 1994; Li and Madanat, 2002; Ouyang and Madanat, 2006*). Therefore, the cyclic time interval between two rehabilitation activities is used as the analysis period. Figure 6-2 gives an example showing different life cycles for two paving strategy alternatives (Alt 1 and Alt 2).

[3] A stochastic process has the **Markov property** if the conditional probability distribution of future states of the process depend only upon the present state; that is, given the present, the future does not depend on the past.

Uncertainties and stochastic features in the deterioration process are beyond the scope of this study.



(a) Cycle Length of Alt 1 (a) Cycle Length of Alt 2

Figure 6-2 Pavement Strategies and the Corresponding Cycle Length

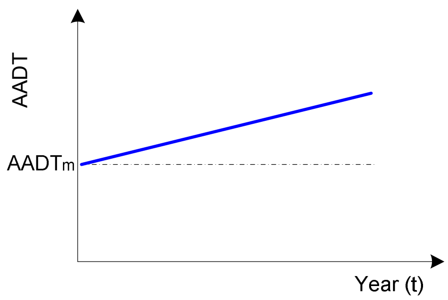


Figure 6-3 Traffic Volume Trend

It is assumed here that the traffic expected to travel over the pavement increases linearly over time with a constant growth rate (GR), as shown in Figure 6-3. Note that the GR refers to actual growth rate instead of compound growth rate. Annual Average Daily Traffic for future year *t*, represented by *AADT(t)*, can be determined from the baseline $AADT_m$ using Eq.6-3.

$$AADT(t) = AADT_m(1 + GR \cdot t) \qquad \text{Eq.6-3}$$

Pavement is modeled as a two-layer system composed of existing pavement layer and overlay layer.

6.3 Long-Term Pavement Decision

In practice Hot Mix Asphalt (HMA) and Portland Cement Concrete (PCC) overlay over existing pavement are considered to be the most common techniques for rehabilitating existing asphalt and concrete pavements. By placing the needed thickness of paving materials on top of an existing pavement, overlay (resurfacing) will return the pavement to a high level of serviceability and provide the necessary

structural strength for the pavement design period. A candidate paving strategy specifies a set of critical design variables including but not restricted to:

(1) the type of paving materials;

(2) the thickness of the layers;

(3) the restored serviceability level (P_m);

(4) the threshold serviceability level (P_t);

(5) the necessary annual maintenance cost (C_{LM}).

6.3.1 Life Cycle Length

Given necessary parameters such as traffic loading, material qualities and environment conditions, the expected length of time interval between periodic rehabilitation actives, called life cycle length (CL), can be predicted by using the AASHTO 1993 design equation inversely (*AASHTO, 1993*). In the 1993 AASHTO Guide, the design equations for flexible pavements and rigid pavements are as follows.

$$\log_{10}(W_{18}) = A \qquad \text{Eq.6-4}$$

For Flexible Pavements

$$A = Z_R \times S_0 + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.40 + \frac{1094}{(SN+1)^{5.19}}}$$
$$+ 2.32 \times \log_{10}(M_R) - 8.07$$

For Rigid Pavements

$$A = Z_R \cdot S_0 + 7.35 \cdot \log_{10}(Th+1) - 0.06 + \frac{\log_{10}\left(\frac{\Delta PSI}{4.5-1.5}\right)}{1+\frac{1.64\times10^7}{(Th+1)^{8.46}}}$$

$$+ (4.22 - 0.32 \cdot p_t) \cdot \log_{10}\left[\frac{S_c^{'} \cdot C_d \cdot (Th^{0.75} - 1.132)}{215.63 \cdot J \cdot \left[Th^{0.75} - \frac{18.42}{(E_c / k_m)^{0.25}}\right]}\right]$$

where, W_{18} = predicted number of 18 kip Equivalent Single Axle Load (ESAL);
Z_R = standard normal deviate;
S_0 = combined standard error of the traffic prediction and performance prediction;
M_R = subgrade resilient modulus (in psi);
ΔPSI = difference between the restored design serviceability index and threshold serviceability index, $\Delta PSI = P_m - P_t$;
SN = structure number, $SN = \sum_{i=0}^{N_L} a_i m_i Th_i$
- N_L =the number of layers;
- a_i =the i^{th} layer coefficient;
- m_i =the i^{th} layer drainage coefficient;
- Th_i =the i^{th} layer thickness (in.);

Th = thickness of pavement slab (in.);
$S_c^{'}$ = modulus of rupture of PCC(psi);
C_d = drainage coefficient;
J = load transfer coefficient;
E_c = modulus of elasticity of PCC (psi);
k_m = modulus of subgrade reaction (lb/in.^{3});

Assuming that the traffic grows linearly over time, the accumulated number of ESALs over the life cycle can be calculated using Eq.6-5

$$W_{18} = 365 \cdot CL \cdot \left[\frac{AADT_m + AADT_m(1 + GR \cdot CL)}{2}\right] \cdot F_{ESAL} \cdot D_L \qquad \text{Eq.6-5}$$

where, CL = life cycle length (yr);
GR = traffic growth rate;
$AADT_m$ = designed AADT for baseline year when rehabilitation is conducted ;
D_L = lane distribution factor;
F_{ESAL} = ESAL factor, $F_{ESAL} = \sum_{i=1}^{NV} PV_i EV_i$;
- NV =the number of vehicle classes;
- PV_i =percentage of the i^{th} vehicle class;
- EV_i =ESAL factor of the i^{th} vehicle classs;

We substitute Eq.6-5 into Eq.6-4 and obtain the predicted *CL* by solving the resulting quadratic equation.

$$CL = \frac{1}{GR}\left(\sqrt{1 + 2 \cdot GR \cdot \frac{10^A}{B}} - 1\right) \qquad \text{Eq.6-6}$$

where, $B = 365 \cdot AADT_m \cdot F_{ESAL} \cdot D_L$

6.3.2 Maintenance Time and Cost

Generally, thinker overlay and improved materials would achieve longer life cycle length. However, the associated time and cost required to accomplish unit resurfacing work are expected to increase due to higher material prize, increased amount of material and intensity of work activity (the size and number of the equipment and labor needed in work zones). Therefore, in addition to the resulting life cycle length, candidate paving strategies may differ in terms of the following two parameters:

(1) The average agency cost required to accomplish a unit amount of rehabilitation work, represented by the parameter z_2;

(2) The average work time needed to complete a unit amount of work, represented by the parameter z_4.

For instance, thin HMA overlays (2 inches and less thickness), medium HMA overlays (2-4 inches thickness), and thick HMA (4 inches and greater thickness) overlays usually extend the pavement serviceability 5-10 years, 8-12 years, and 10-15 years, respectively. The average agency costs for overlays with different thicknesses can range from \$0.07 million/lane-mile to \$0.48million/lane-mile (*NYSDOT, 2006*). According to data obtained from the Maryland State Highway Administration, it takes

about 4 hours to pave one lane-mile pavement with 1.5 inches HMA overlays and 5.2 hours per lane mile for 2.0 inch HMA overlays.

Since the overlay thickness (*D*) is one of the most significant design variables when the material type is determined, it is assumed that the unit work time and cost, z_2 and z_4, increase linearly with the overlay thickness, as expressed in Eq.6-7 and Eq.6-8, in which a_2, b_2, a_4, *and* b_4 are parameters that can be obtained from field data or through regression analysis based on historical database.

$$z_2 = a_2 + b_2 \cdot Th \qquad \text{Eq.6-7}$$

$$z_4 = a_4 + b_4 \cdot Th \qquad \text{Eq.6-8}$$

where, Th = overlay/slab thickness (in.)

6.4 Cost-Effectiveness Index

To economically evacuate a number of feasible pavement design alternatives and identify one that may be the most cost-effective to build and maintain, a Cost-Effectiveness Index (CEI) is developed to consider user benefit attained with relation to long-time riding qualities and greater durability as well as the total cost to the agency and to the road users over the pavement life cycle.

The effectiveness indicator is based on the average product of AADT and pavement condition index over the life cycle. The cost indicator is the EUAC per lane mile. The ratio of the effectiveness indicator to the cost indicator is used as the measure of cost-effectiveness of candidate pavement strategies.

$$CEI = \frac{EI}{CI} = \frac{\frac{1}{CL}\int_0^{CL} AADT(t) \cdot P(t) \cdot dt}{C_{CL}} \qquad \text{Eq.6-9}$$

where, CEI = Cost-Effective Index;
CI = Cost Indicator;

EI = Effectiveness Indicator;
t = Age (years);
$P(t)$ = Pavement condition for year t;
$AADT(t)$ = Annual Average Daily Traffic Volume for year t;
C_{CL} = EUAC per lane-mile (\$/lane-mile).

6.4.1 Effectiveness Indicator

The variation of the pavement condition with age, *P(t)*, is modeled with a deterministic pavement deterioration model. The pavement surface roughness or riding comfort is represented by the Present Serviceability Index (PSI). It is a scale between 0 and 5, with 5 being a perfectly smooth ride and 0 being an essentially impassable pavement. In this study, the deterioration curve for a rehabilitated pavement is expressed as:

$$P(t) = P_m - b \cdot t^a \qquad \text{Eq.6-10}$$

where, $P(t)$ =PSI at time t;
a = a constant parameter which controls the degrees of curvature of the performance curve;
b =a constant chosen in such a way that $P(CL)=P_t$, $b = \dfrac{P_m - P_t}{CL^a}$.

By substituting Eq.6-3 for *AADT(t)* with Eq.6-10 for *P(t)*, the effectiveness indicator (EI) can be derived as follows:

$$\begin{aligned} EI &= \frac{1}{CL}\int_0^{CL} AADT(t) \cdot P(t) \cdot dt \\ &= \frac{1}{CL}\int_0^{CL} AADT_m \cdot (1 + GR \cdot t) \cdot (P_m - b \cdot t^a) \cdot dt \\ &= AADT_m \cdot [(P_m - \frac{P_m - P_t}{a+1}) + GR \cdot (\frac{P_m}{2} - \frac{P_m - P_t}{a+2}) \cdot CL] \end{aligned} \qquad \text{Eq.6-11}$$

6.4.2 Cost Indicator

Competing pavement strategies may result in different life cycle durations (*CL*). Therefore, EUAC (per unit area), which produces the yearly costs of an alternative as if they occurred uniformly throughout the analysis period, is used to compare the life

cycle costs associated with each competing alternative. The cost components in this analysis include:

(1) One-time Rehabilitation Costs (C_T)

One-time Rehabilitation Costs consist of those agency costs (C_A) and user costs (C_U) associated with rehabilitation of a pavement at time 0 in a periodic life cycle. Detailed formulations of each cost components can be found in Chapter 3. The value of one-time rehabilitation costs depends highly on the time/cost parameter (z_2 and z_4), traffic condition ($AADT_m$ and distribution), and short-term work zone decision.

(2) Long-term Future Costs (C_{LT})

In this analysis, a constant annual maintenance cost per lane mile (C_{LM}) is considered as the major component of the long-term future costs. This cost is the expenditure by transportation agencies on annual routing maintenance activities such as crack sealing and pothole patching. Long-term user costs in normal operations, mainly vehicle operating cost associated with the consumption of fuel and tire and discomfort due to road roughness are excluded in the cost indicator. The effect of paving strategy on traffic is considered instead in the benefit indicator.

Therefore, EUAC per lane-mile can be calculated from the following equation, where L_T represents the total lane-mile of the pavement to be rehabilitated.

$$CI = C_{CL} = \frac{1}{L_T}\left[C_T \cdot \frac{i(1+i)^{CL}}{(1+i)^{CL}-1}\right] + C_{LM} \qquad \text{Eq.6-12}$$

6.5 Joint Optimization of Long-Term and Short-term Decisions

The cost-effectiveness of a paving strategy can be affected by one-time rehabilitation costs, which significantly vary with short-term work zone decision. On the other hand, the most suitable short-term work zone decisions depend on which paving strategy is utilized accounting for different unit maintenance time and cost associated with each strategy. Consequently, it will aid the agencies to make the most cost-effective investment from a long-term point of view by jointly optimizing the long-term pavement decisions and short-term work zone decisions.

With the objective of gaining the highest Cost-Effectiveness-Index (CEI), the index of the candidate pavement strategies along with the work zone decision variables considered in Chapter 3 are jointly optimized. The optimization model is expressed as follows:

Model 2

<u>Objective:</u>

$$Max \quad CEI(k, m, \vec{X}) = \frac{EI}{CI}$$

$$= \frac{AADT_m \cdot [P_m(k) - \frac{P_m(k) - P_t(k)}{a+1} + GR \cdot (\frac{P_m(k)}{2} - \frac{P_m(k) - P_t(k)}{a+2}) \cdot CL(k)]}{\frac{1}{L_T}\left[C_T(k) \cdot \frac{i(1+i)^{CL(k)}}{(1+i)^{CL(k)} - 1}\right] + C_{LM}(k)}$$

<u>Subject to:</u>

(1) Rehabilitation Budge Constraint

(2) Work Zone Operation and Traffic Impact Constraints

where, k =the index of the paving strategy alternative ID;

m =Number of work zones;

$\vec{X}$ =Work zone characteristics.

L_T =Total lane-mile of the pavement required to be rehabilitated.

Figure 6-4 illustrates the procedure to solve this optimization problem. For each candidate pavement, the effectiveness indicator is calculated with Eq.6-11 and then the cost indicator is obtained by calling the short-term work zone decision optimization model developed in Chapter 3.

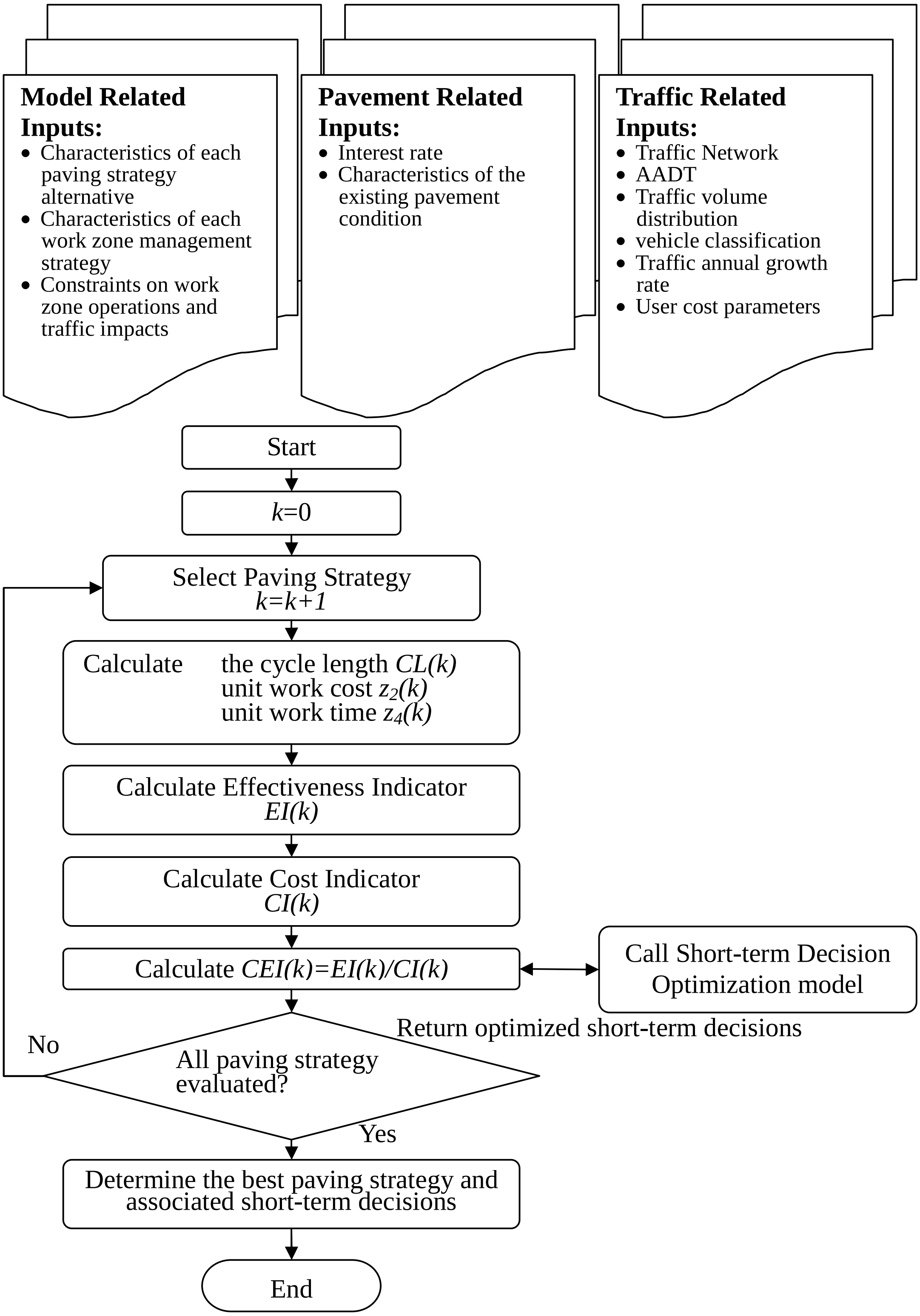


Figure 6-4 Procedure of Joint Optimization of Long-Term and Short-Term Decisions

6.6 Numerical Examples

6.6.1 Experiment Design

To examine the capability of the proposed joint optimization methodology, a set of paving strategy alternatives are provided for consideration in the sample maintenance project on a 4-lane freeway section with an alternative route studied in Chapter 4.

In all strategies one layer of HMA overlay is placed on the existing pavement that has a residual structure number (SN) of 2.6 and the overlay restores the pavement PSI from 2.5 to 4.5. The annual routine maintenance cost is 1000$ per lane-mile. These strategies differ in overlay thicknesses ranging from 1.5 inches to 5 inches, which leads to the variation of associated unit rehabilitation time, unit rehabilitation cost, and the life cycle duration.

Table 6-1 Description of the Candidate Paving Strategies

Paving Strategy	Strategy Description	Category	Overlay Thickness	P_m	P_t
#			*inch*		
1	Thin HMA Overlay	Light Rehabilitation	1.5	4.5	2.5
2	Thin HMA Overlay	Light Rehabilitation	2.0	4.5	2.5
3	Medium HMA Overlay	Moderate Rehabilitation	2.5	4.5	2.5
4	Medium HMA Overlay	Moderate Rehabilitation	3.0	4.5	2.5
5	Medium HMA Overlay	Moderate Rehabilitation	3.5	4.5	2.5
6	Thick HMA Overlay	Heavy Rehabilitation	4.0	4.5	2.5
7	Thick HMA Overlay	Heavy Rehabilitation	4.5	4.5	2.5

Assuming a fixed cost of 30,000 $/lane-mile and a variable cost of 52,000 $/lane-mile per inch, the unit rehabilitation cost can be derived from the following linear function:

$$z_2 = 30{,}000 + 52{,}000 \cdot Th$$

where, Th = thickness of HMA overlay (inch);
z_2 = unit rehabilitation cost ($/lane-mile);

Based on the data obtained from the Maryland State Highway Administration (*Chen, 2004*), the following linear regression models are developed to estimate the unit rehabilitation time given the thickness of HMA overlay.

$$z_4 = 0.4 + 2.4 \cdot Th$$

where, z_4 = unit rehabilitation time (hr/lane-mile);

Table 6-2 provides the cost and time information associated with each test paving strategy. The variations of the unit rehabilitation cost and time with the HMA overlay thickness are illustrated in Figure 6-5 and Figure 6-6. Given the necessary pavement design parameters listed in

Table 6-3, the life cycle length can be predicted with Eq.6-6 once $AADT_m$ and other traffic related information is available. Figure 6-7 illustrates the change of the life cycle duration (CL) with increasing overlay thickness under different $AADT_m$ levels.

To investigate the effects of traffic level, the short-term work zone decisions and long-term pavement decisions will be jointly optimized under the AADT ranging from 60,000 to 140,000, at increments of 20,000 vehicles/day. The traffic distribution is fixed. An AADT of 100,000 vehicles/day is set as the baseline scenario. Another important experiment tests whether jointly optimization is necessary or not. Since detour control has the potential to significantly affect the one-time work zone cost, the joint optimization is conducted twice for each AADT level scenario: one without any detour control and another with SO detour control.

Sensitivity analysis is conducted to examine how variations in traffic growth rate and discount rate affect the optimization results.

Table 6-2 Cost Information of the Candidate Pavement Strategies

Paving strategy	z_1	z_2	z_3	z_4	*CLM*
#	$/lm	$/lm	hr/lm	hr/lm	$/lm-year
1	1000	78000	2	8.0	1000
2	1000	94000	2	10.4	1000
3	2000	110000	4	12.8	1000
4	2000	126000	4	15.2	1000
5	2000	142000	4	17.6	1000
6	4000	158000	6	20.0	1000
7	4000	174000	6	22.4	1000

*$/lm=dollars per lane-mile; hr/lm=hours per lane-mile; $/lm-year=dollars per lane-mile per year

Table 6-3 Pavement Design Parameters

Variable	Description	Value	
Material	Overlay material	HMA	
Z_R	standard normal deviate	0.40	
S_0	combined standard error of the traffic prediction and performance prediction	0.35	
M_R	subgrade resilient modulus (psi)	10,000	
N_L	the number of new layers	1	
a_i	the layer coefficient	0.26	
m_i	the layer drainage coefficient	0.9	
SN_0	Structure number of the existing pavement	2.6	
E_1	ESAL factor for passenger cars	0.0002	
E_2	ESAL factor for Single Unit Trucks	0.37	
GR	Baseline Traffic Growth Rate	3%	
i	Baseline Interest Rate	4%	
N	the number of lanes in the maintained road section	4	
D_L	Lane Distribution Factors	100%	(*N*=1)
		90%	(*N*=2)
		70%	(*N*=3)
		60%	($N \geq 4$)

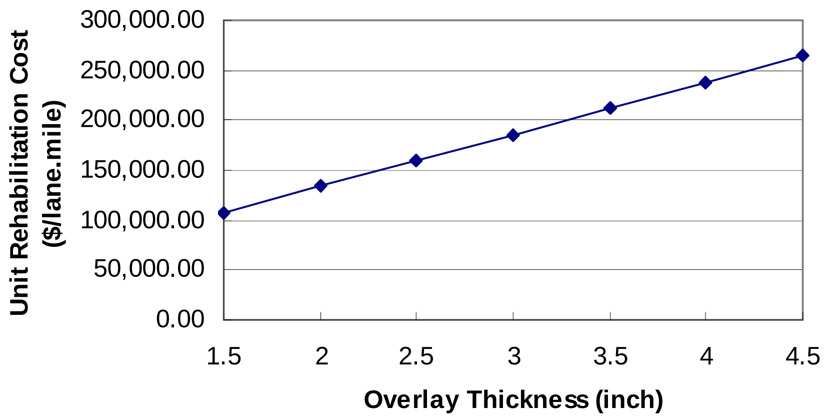


Figure 6-5 Variation of Unit Rehabilitation Cost with Overlay Thickness

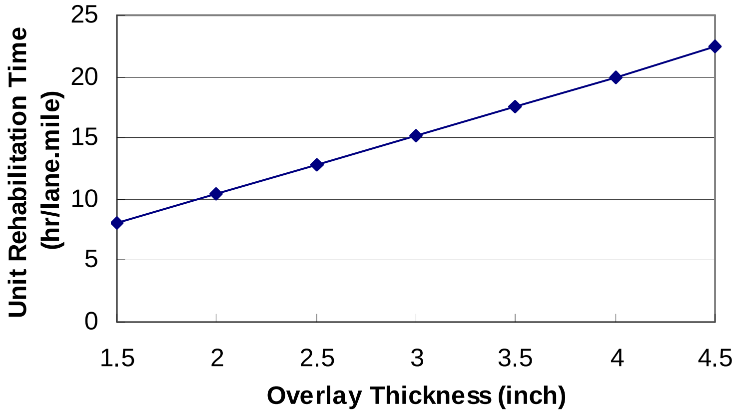


Figure 6-6 Variation of Unit Rehabilitation Time with Overlay Thickness

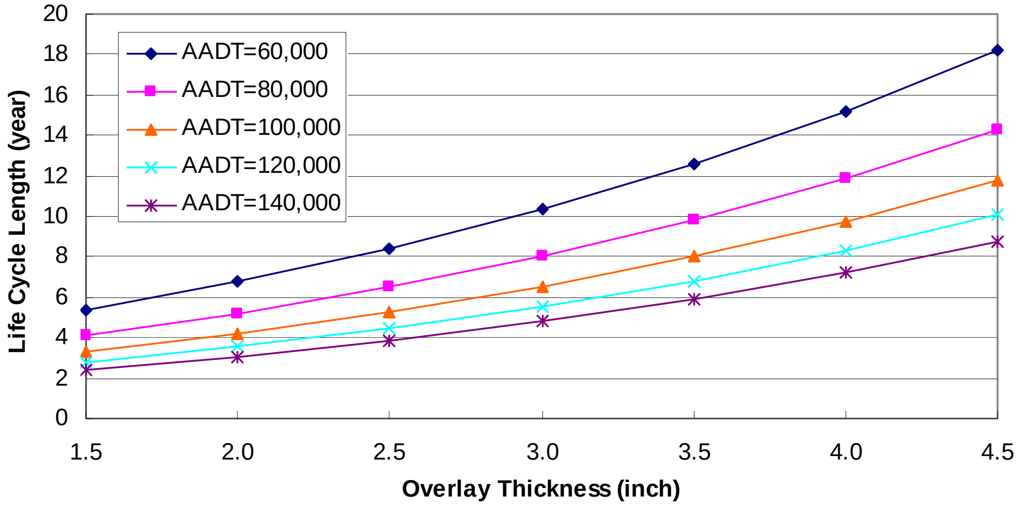


Figure 6-7 Variation of CL with Overlay Thickness at Different Traffic Levels

6.6.2 Optimization Results

(1) No Detour Control in Work Zones

Without using any detour control, the jointly optimized long-term and short-term decisions at varying AADT levels are provided in Table 6-4 and Table 6-5. The variation of Effectiveness Index, Cost Index, and the optimization objective CEI with pavement thickness and traffic level is shown in Figure 6-8. It can be seen that the

paving strategy with the highest CEI and that with the lowest CI may be different. At lower traffic levels (AADT=60,000 to 80,000), the 4.5 inch overly is the most durable and the most cost-effective paving strategy because temporary work zone activities have less impacts on traffic. At higher traffic levels (AADT=100,000 to 140,000), 4.5 inch overlay results high one-time work zone cost, which leads to high life cycle cost Equivalent Uniform Annual Costs even though it gives the pavement the longest life cycle length. Instead, a 3.5 inch overlay obtains the best tradeoff between EI and CI and is returned as the optimal paving strategy at higher traffic levels.

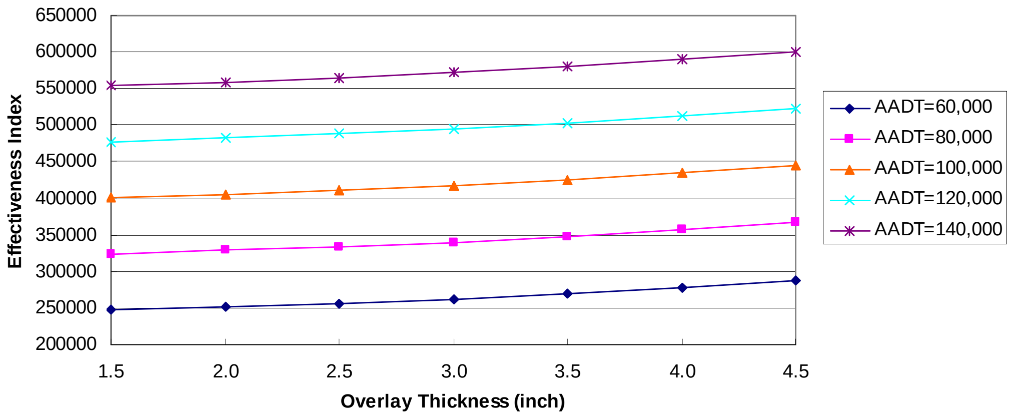
Figure 6-8 (c) indicates that the CEI does not increase or decrease monotonically with thickness. Therefore, it is hard to find a general threshold thickness associated with different traffic levels. This also demonstrates the need for detailed cost-effectiveness analysis when determining long-term work zone decisions.

Table 6-4 Optimized Long-Term Decisions and Long-Term Impacts (No Detour Control)

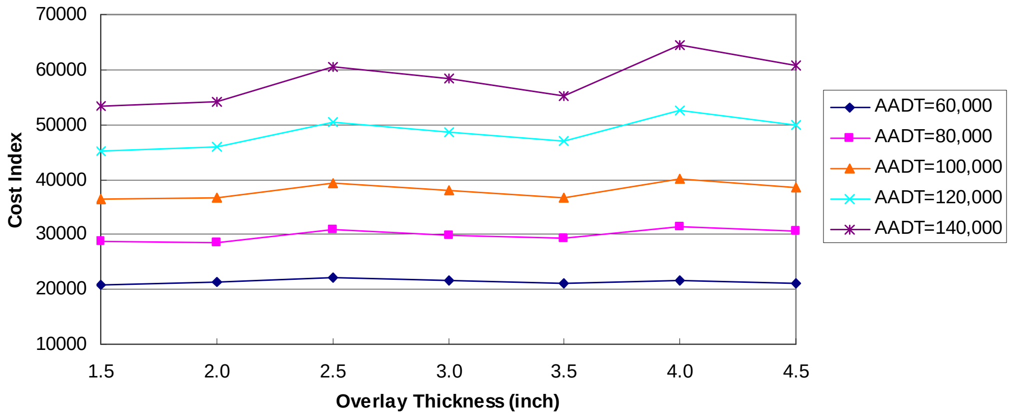
AADT	Optimal Overlay Thickness	Life Cycle Length	EI	CI	CEI
Vehicle/day	*inch*	*year*	-	-	-
60,000	4.5	18.17	287,240.35	21,152.44	13.58
80,000	4.5	14.28	366,656.43	30,570.08	11.99
100,000	3.5	8.03	425,503.73	36,613.86	11.62
120,000	3.5	6.81	502,874.63	46,879.50	10.73
140,000	3.5	5.91	580,072.93	55,121.38	10.52

Table 6-5 Optimized Short-Term Decisions and Short-Term Impacts (No Detour Control)

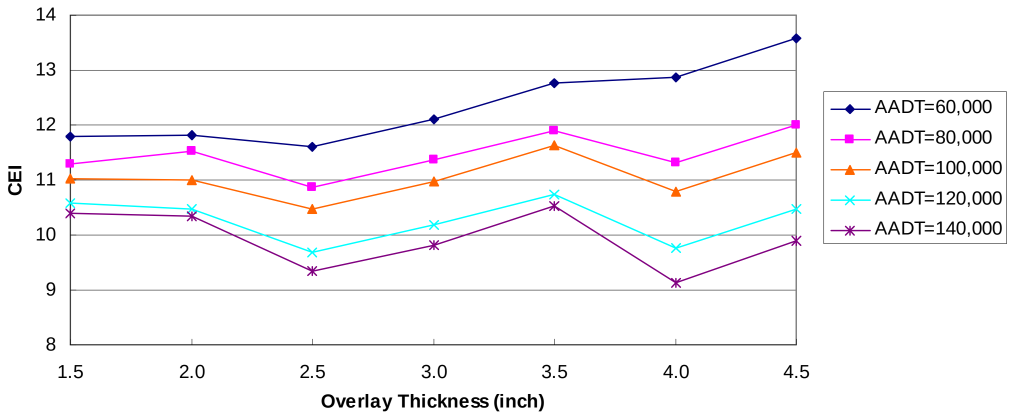
Traffic Level	*# of periods*	*Work Time*	*Zones*	*Str#1*	*Str#2*	*Str#3*	*Str#4*	C_A	C_U	C_T
	#	hr/ period	#	Lane Closure	Work Rate	Detour	Merge Control	$	$	$
60000	26	21	19:00-07:30 07:30-16:00	double single	normal normal	no no	yes no	3,048,210	33,189	3,081,400
80000	35	11	20:00-07:00	double	medium	no	no	3,783,470	21,364	3,804,830
100000	28	10	20:30-06:30	double	normal	no	yes	2,864,420	22,868	2,887,280
120000	30	8.5	21:30-06:00	double	medium	no	yes	3,213,560	10,798	3,224,360
140000	34	8	22:00-06:00	double	medium	no	yes	3,347,200	9,749	3,356,950



(a) EI of each Paving Strategy at varying Traffic Levels (no detour control)



(b) CI of each Paving Strategy at varying Traffic Levels (no detour control)



(c) CEI of each Paving Strategy at varying Traffic Levels (no detour control)

Figure 6-8 Performance Measures of Paving Strategies at varying Traffic Levels (no detour control)

(2) SO Detour Control in Work Zones

Table 6-6 and Table 6-7 provide the jointly optimized long-term and short-term decisions when applying detour control. The variation of Effectiveness Index, Cost Index, and the optimization objective CEI with pavement thickness and traffic level is shown in Figure 6-9. At all traffic levels, 4.5 inch overlay outperforms other pavement options in terms of EI, CI, and CEI.

The test shows that the one-time work zone cost corresponding to each candidate paving strategy can be further reduced by employing efficient traffic impact mitigation strategies, such as merge control and detour control. Since the effectiveness index and life cycle length are not affected by short-term work zone decisions, the cost index (life cycle cost) decreases. Consequently, the cost-effectiveness of heavy rehabilitation is greatly improved at higher traffic levels (AADT=100,000 to 140,000), and the 4.5 inch overlay, instead of 3.5 inch overlay, achieves the highest durability and cost-effectiveness among all alternatives. It clearly indicates that changing short-term work zone decisions does affect long-term paving decisions.

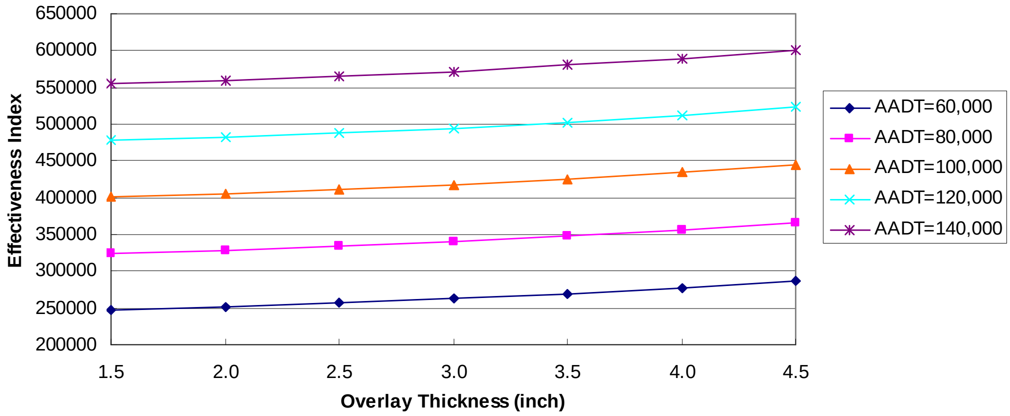
Table 6-6 Optimized Long-Term Decisions and Long-Term Impacts (Detour Control-SO)

AADT	Optimal Overlay Thickness	Life Cycle Length	EI	CI	CEI
Vehicle/da	*inch*	*year*	-	-	-
60,000	4.5	18.17	287,240.35	19,022.42	15.10
80,000	4.5	14.28	366,656.43	23,336.78	15.71
100,000	4.5	11.79	445,229.69	30,505.83	14.59
120,000	4.5	10.05	523,301.78	37,755.58	13.86
140,000	4.5	8.76	601,049.66	45,973.72	13.07

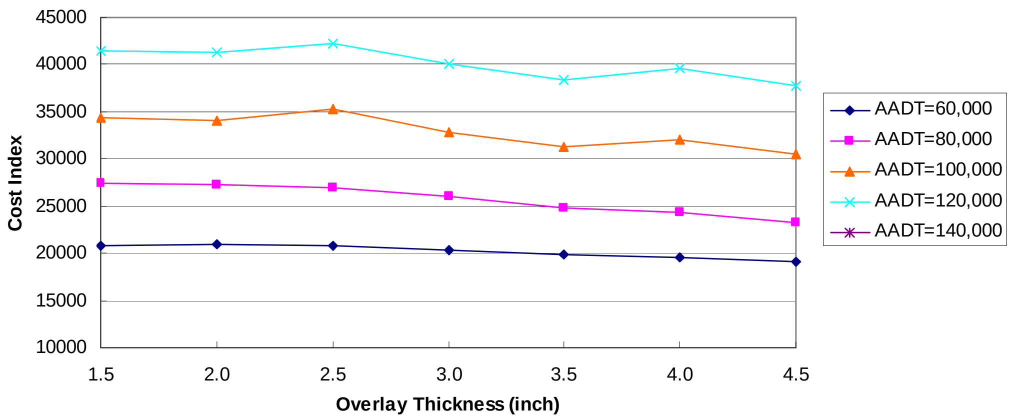
Table 6-7 Optimized Short-Term Decisions and Short-Term Impacts (Detour Control-SO)

Traffic Level	*# of periods*	*Work Time*	*Zones*	*Str#1*	*Str#2*	*Str#3*	*Str#4*	C_A	C_U	C_T
	#	hr/ period	#	Lane Closure	Work Rate	Detour	Merge Control	$	$	$

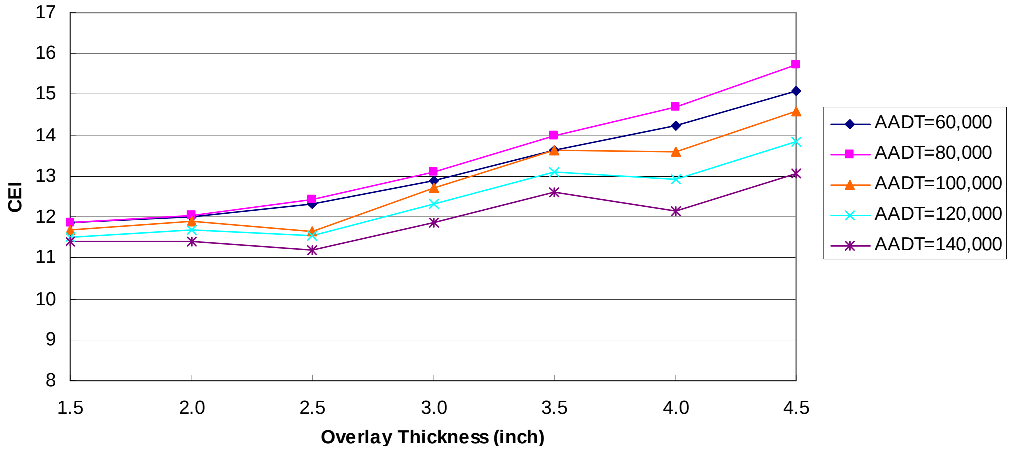
60000	12	24	16:00-16:00	double	normal	SO	yes	2,661,030	94,673	2,755,710
80000	14	22	18:00-16:00	double	normal	SO	yes	2,718,140	155,963	2,874,110
100000	25	21	19:00-08:00	double	normal	SO	yes	3,157,600	119,601	3,277,200
			08:00-16:00	single	normal	SO	yes			
120000	33	12.5	19:00-07:30	double	normal	SO	yes	3,525,970	65,466	3,591,430
140000	35	11	20:00-07:00	double	medium	SO	yes	3,876,360	46,564	3,922,930



(a) EI of each Paving Strategy at varying Traffic Levels (detour control-SO)



(b) CI of each Paving Strategy at varying Traffic Levels (detour control-SO)



(c) CEI of each Paving Strategy at varying Traffic Levels (detour control-SO)

Figure 6-9 Performance Measures of each Paving Strategies at varying Traffic Levels (detour control-SO)

6.6.3 Sensitivity Analysis

(1) Interest Rate (*i*)

The interest rate (*i*) is the discount rate by which future costs will be converted to present value. It reflects the opportunity value of time and may vary with the degree of risk and uncertainty. By increasing interest rate from 4% to 10%, the optimal paving strategy in the baseline scenario (AADT=100,000, no detour control) switches from 3.5 inch overlay to 1.5 inch overlay, as shown in Figure 6-10. The performance measure indices (EI, CI and CEI) associated with each paving strategy under interest rate of 4% and 7% are compared in Figure 6-11 (a), (b), (c). It shows that a higher interest rate does not affect the EI but increases the CI measured by EUAC. When the paving strategy is ranked based on CEI value, high interest rates favor a low intensity rehabilitation option with smaller capital investment.

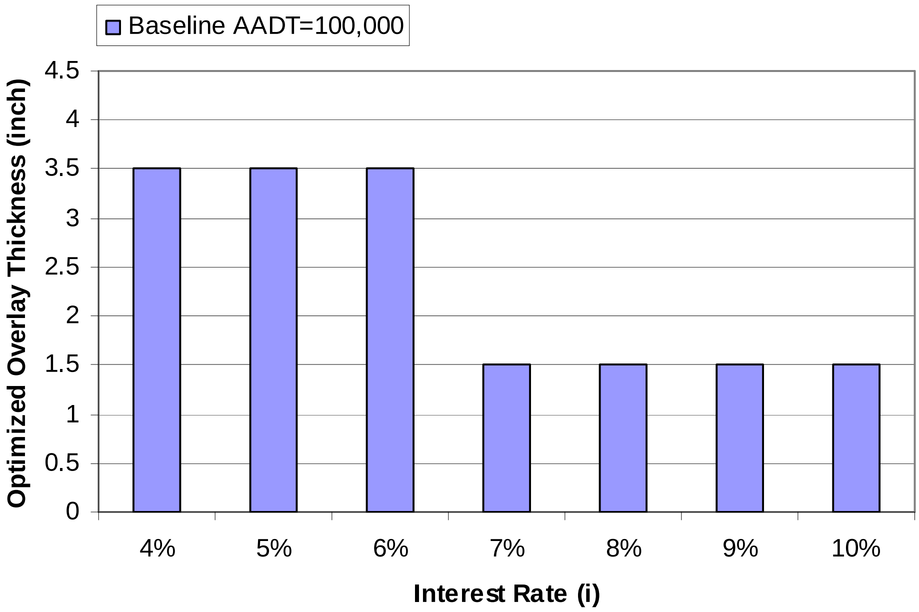
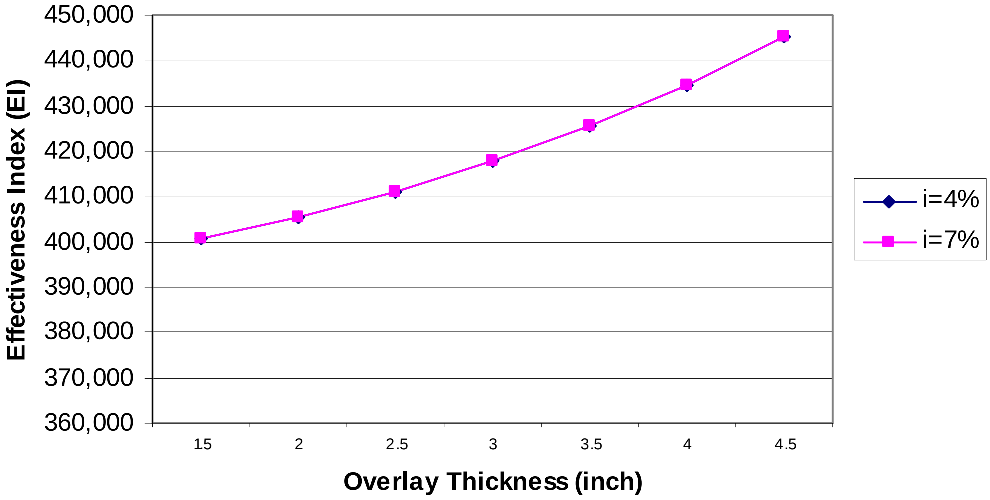
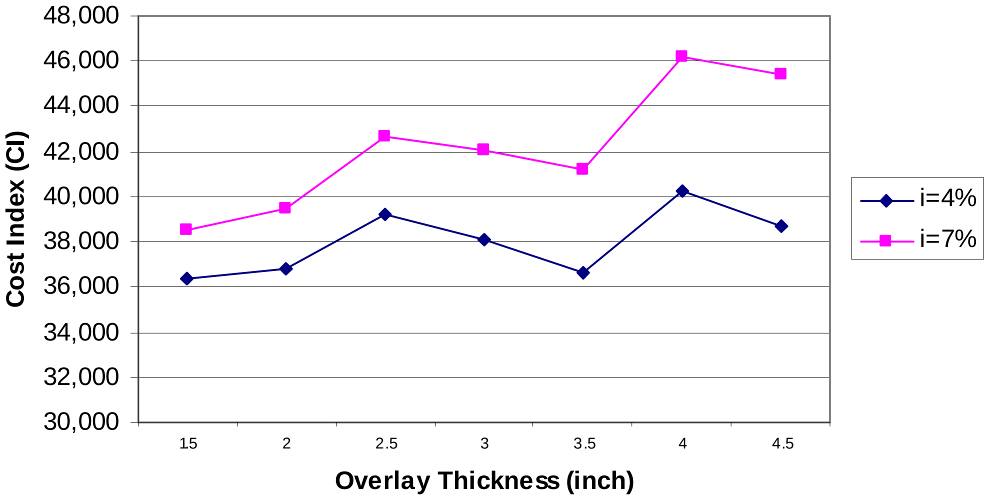


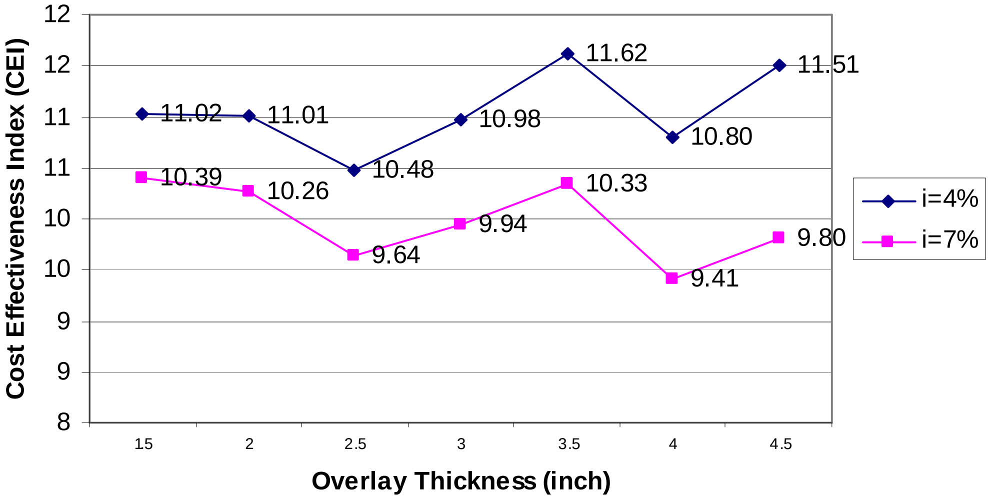
Figure 6-10 Optimized Paving Strategy for Varying Interest Rates



(a) EI of each Paving Strategy for Varying Interest Rates



(b) CI of each Paving Strategy for Varying Interest Rates



(c) CEI of each Paving Strategy for Varying Interest Rates

Figure 6-11 Performance Measures of each Paving Strategy for varying Interest Rates

(2) Traffic Growth Rate (GR)

Traffic growth rate (GR) is an important parameter in calculating pavement life cycle duration, as expressed in Eq.6-6. The GR reflects the future traffic loads. Figure 6-12 gives the optimal overlay thickness when the GR varies from 3% to 9%. When GR is below 7%, 3.5 inch overlay is the most cost-effective strategy. After GR exceeds 7%, thicker overlay (4.5 inch) outperforms the other options. In Figure 6-13, the performance measure indices (EI, CI and CEI) associated with each paving strategy in

baseline scenario given 3% and 7% GR are compared. It can be seen that both EI and CI of a paving strategy increase when GR rises. The variation of GR doesn't change the one-time work zone cost but affect the life cycle length. Therefore, the CI of a given paving strategy increases with a higher GR. Since more travelers will be served in future year, the EI also increases. The new CEI, as ratio of increased EI and CI, may change the ranking of paving strategies, as shown in Figure 6-13 (c). Generally, high intensity rehabilitation is preferable at high traffic growth rates because it can provide better paving conditions for future traffic.

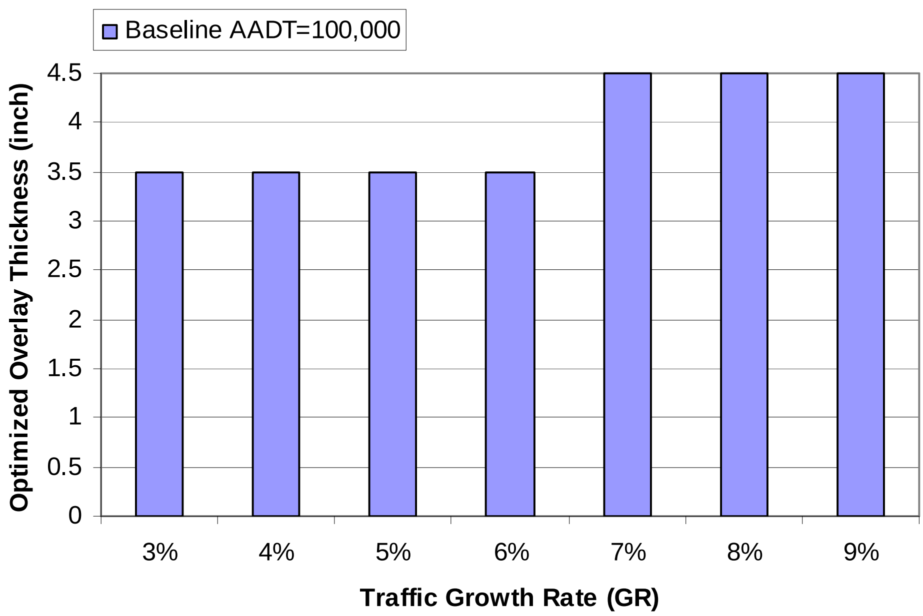
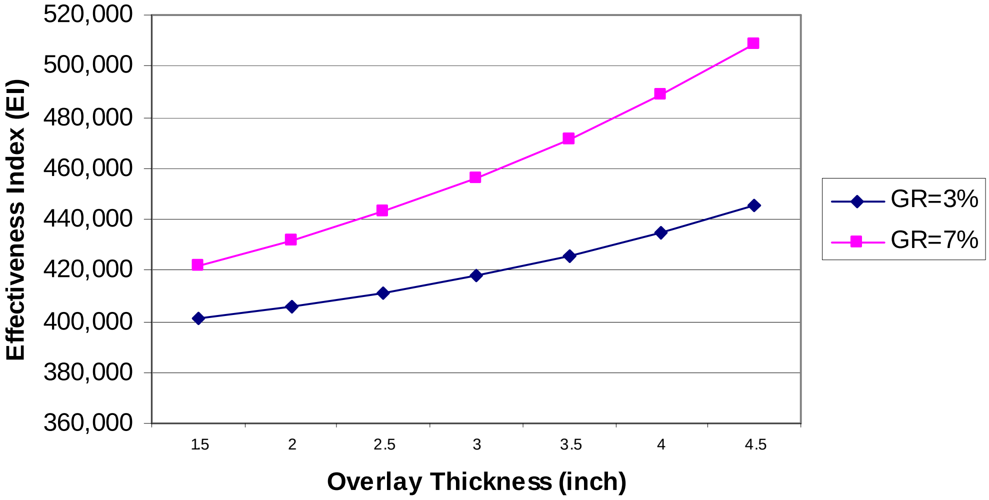
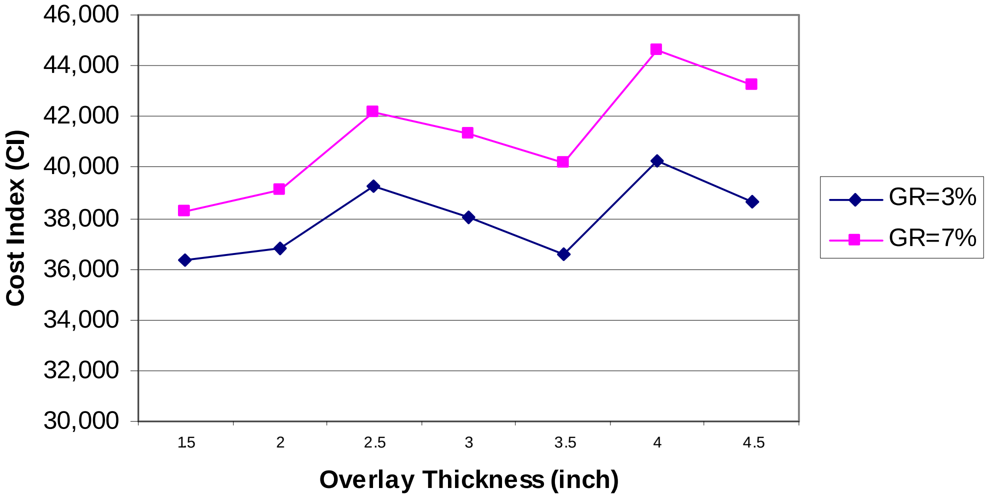


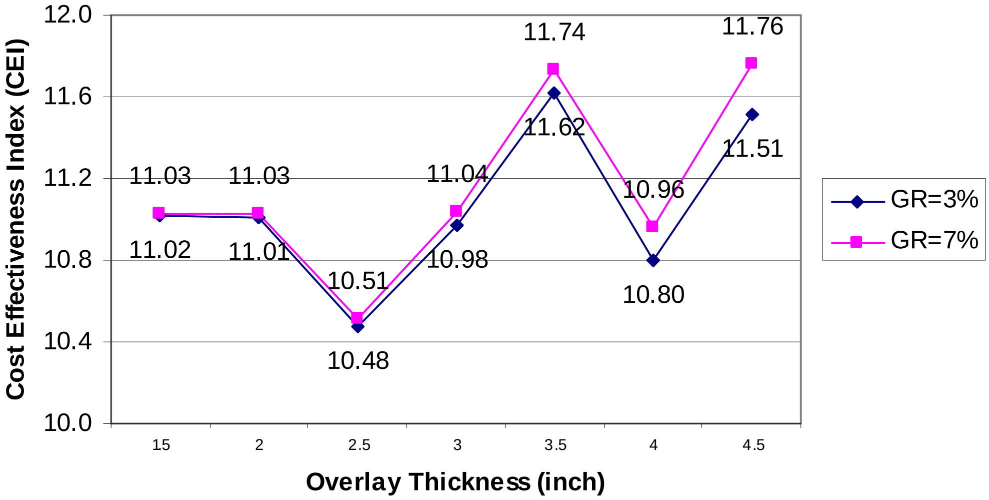
Figure 6-12 Optimized Paving Strategy for Varying Traffic Growth Rates



(a) EI of each Paving Strategy for Varying Traffic Growth Rates



(b) CI of each Paving Strategy for Varying Traffic Growth Rates



(C) CEI of each Paving Strategy for Varying Traffic Growth Rates

Figure 6-13 Performance Measures of each Paving Strategy for varying Traffic Growth Rate

6.6.4 Findings

The extensive experiments demonstrate that the most "desirable" paving strategy may vary, depends on whether long-term user benefit (measured by EI) is taken into account, and whether short-term work zone decisions are optimized. With the objective of maximizing the Cost-Effectiveness Index, which reflects the tradeoff between benefits and costs, the proposed jointly optimization model can obtain a maintenance plan with satisfactory durability and relatively low life cycle cost.

The optimized long-term and short-term decisions are sensitive to the variation of current traffic level, traffic growth rate, and interest rate. A high intensity paving strategy usually achieves the longest performance life but the associated high one-time work zone cost may affect its cost-effectiveness especially at high traffic levels. Employing efficient traffic management tactics and optimizing short-term work zone decision are useful in reducing the life cycle cost. High traffic growth rates favor higher intensity strategy while high interest rates favor the opposite strategies.

Chapter 7 Conclusions and Future Work

7.1 Summary of the Dissertation

A significant fraction of the nation's current highway system has been in poor, mediocre, or fair condition. To sustain highways in a safe and usable condition, state and federal transportation agencies have increased the number, duration, and scope of maintenance activities in recent years. However, roadway capacity reduction caused by work zone activities is one of the principal contributors to non-recurrent congestion, and the resulting disruption to traffic may significantly affect the safety of the travelling public, workers and other highway users. To address the above issue, the Federal Highway Administration has promoted the concept of a comprehensive transportation management plan (TMP), which is an innovative combination of various coordinated mitigation strategies encompassing construction plans, temporary traffic control (TTC) measures, traffic operation management, safety management, and other factors. (*FHWA, 2005*). All federal, state, and local highway agencies are encouraged to develop and implement TMPs for planned work zone activities. With the public's increasing need for better mobility and safety in work zones, TMP design remains a challenging task for agencies, especially when the scale and complexity of the project increase.

To aid decision makers design an effective TMP, a comprehensive methodology has been developed here for evaluating as well as optimizing work zone decisions. After examining major concerns of involved stakeholders and the state of art research and practice on work zone management in Chapter 2, a short-term work zone impact

evaluation model is established in Chapter 3. This analytical model calculates the resulting one-time work zone cost given a set of critical work zone characteristics. In Chapter 4, a work zone decision optimization model is proposed to minimize short-term impacts. A Two-Stage Population Based Simulated Annealing (2PBSA) algorithm is developed to search for near-optimal solution to the proposed optimization problem. The capability of the optimization method is demonstrated through numerical experiments in which the analytical one-time work zone cost model is used to evaluate the objective function value of each candidate solution. Chapter 5 explores the feasibility of conducting short-term work zone decision optimization with the solution evaluated by simulation instead of analytical model. Since simulation-based simulation is quite consuming, two speed-up methods, hybrid method and parallel computing, are proposed to accelerate the optimization process. Chapter 6 proposes a model to jointly optimize short-term and long-term work zone decisions which can maximize the cost-effectiveness, quantified by an index reflecting the cost-benefit trade-off.

7.2 Research Contributions

The main contributions of this research are described as follows:

(1) Developing a systematic approach integrating short-term and long-term work zone decisions into one optimization framework

In previous work, the selection of long-term paving strategies was usually made by experts on paving management while the optimization of work zone management plans was mostly explored by researchers in the field of traffic engineering. Their works are relatively separate due to different study scopes. However, those long-term

and short-term maintenance decisions are highly interrelated in practice. In this research a methodology is proposed with which decisions on paving and traffic parts are jointly optimized so that improved cost-effectiveness can be achieved.

To further reduce the work zone impacts on road users, highway workers, businesses, and community, nowadays it is a rising trend of encouraging designers/construction engineers, traffic engineers, safety experts and other technical specialists are increasingly encouraged to work together on developing a comprehensive work zone management plans employing a set of coordinated mitigation strategies. The methodology developed in this dissertation overcomes the limitations of repetitive manual process required in traditional work zone management plan design, so that high-quality optimized design sets can be provided to decision makers.

(2) Improving the analytic model to estimate short-term work zone impacts

To make wise work zone decisions, it is important to precisely evaluate their impacts on traffic mobility and safety. A number of analytic models have been proposed in previous studies. However, most of them are based on deterministic queuing models and do not consider the stochastic nature of traffic flow. In addition, over-simplified assumptions on work zone speed estimation or detour behavior further limit the accuracy of their estimation results. The analytic model developed in this study takes account of the effect of the traffic randomness and other delays which are difficult to model analytically by introducing a work zone systematic delay. A regression model derived from simulation results is established to estimate systematic delay. Instead of using a pre-determined fixed value, the average work zone speed is time-varying and it

is estimated with a speed-flow model also developed from simulation results. A modified Bureau of Public Roads (BPR) function proposed by Skabardonis and Dowling (*1997*) is used to estimate delays on detour routes because its results better fit the real-world data and simulation results than the standard BPR function, especially when volume-to-capacity ratios exceed 1.0.

An experiment comparing the performances of the proposed analytical model and CORSIM simulation model shows that the analytical model is able to quickly estimate work zone delay at a satisfactory precision if important input data, such as work zone capacity and work zone speed, are accurately provided.

(3) Incorporating different detour models

Diverting traffic to alternative route has great potential to mitigate the traffic impact by utilizing spare capacity in a road network. However, the drivers' diversion behavior responding to lane closure was not well modeled in previous studies. This research embeds three models for predicting time-varying detour rates:

- System Optimization (SO) model returns the optimal diversion rate which minimizes the total delay on mainline and detour routes within each time unit. The SO diversion rates obtained from the SO model can provide control objectives for Intelligent Transportation System (ITS) in work zone sites.
- Logit-based Route Choice (RC) model borrowed from Song and Yin (*2008*) estimates diversion rates based on the difference of travel time on mainline and detour routes. The RC model is more suitable for road users who are not familiar

with the work zone situation and are quite sensitive to travel time information provided by ITS.

- User Equilibrium (UE) model obtains the diversion rate which minimizes the difference in travel time between mainline and detour routes. The UE model is recommended for situations in which road users have relatively good knowledge of the road network and traffic conditions.

Through numerical experiments, we found that selection of detour model considerably affects the optimized work zone decisions. Therefore, before applying our work zone impact estimation model in real-world projects, it is critical to select an appropriate detour model based on data from stated-preference surveys or other user behavior studies.

(4) Introducing traffic impact mitigation strategies into optimization model

Most of existing work zone optimization studies developed their models based on selected work zone decisions, which may restricts the model extendibility. In this research, we proposed a one-time work zone cost minimization model to optimize work zone decisions that may be frequently considered in a TMP. The decision variables are classified into continuous decision variables and discrete ones. Lane closure starting and ending times are included as continuous decision variables. A data type called "general strategy" is created to flexibly model any discrete decision variable. By this means, it would be quite convenient to add into the model customized

traffic impact mitigation strategies (e.g. lane closure type, accelerated work rate, capacity adjustment, demand adjustment, etc.) as decisions to be optimized.

(5) Developing the optimization model for recurrent work zones

We noticed that lane closure schedules in most major maintenance projects are periodic time windows due to repeated daily or weekly patterns exhibited by the traffic. However, previous studies on work zone optimization do not fully utilize this characteristic and as a result their problem size and complexity increases with the maximal allowable project duration. This may potentially reduce the efficiency and scalability of their proposed search algorithm.

In addition to an optimization model for general work zones (Model 1-1) and a simplified optimization model for a single work zone (Model 1-3), another model (Model 1-2) modified from the general model is formulated to optimize work zone decisions in a user-specified cyclic period, such as a weekday or a weekend. The major assumption of this model is that the same work zone operation repeats until the maintenance task is completed. By considering theses practical issues, we expect that the gap between theory and practice can be reduced and the flexibility and scalability of the optimization procedure can be improved.

(6) Developing an efficient algorithm to solve the optimization problem

A heuristic algorithm, called the two-stage population-based simulated annealing algorithm (2PBSA), is developed to solve the short-term work zone optimization problem. In the first stage of this algorithm, an initial optimization process focusing on optimizing continuous decision variables is performed and the result is then sent to

the second stage as a relatively good initial solution. In the second stage, a refined search focusing on discrete decision variables is conducted inside the promising region provided by the first step.

In a test problem, the result of the convergence, optimality and reliability analysis proves that the 2PBSA reliably obtains near-optimal solutions. Since low traffic levels may enlarge the feasible solution space, it is recommended that the optimized solution quality should be ensured by increasing the population size at the first stage and the number of generations at the second stage.

(7) Developing a simulation-based optimization model

Microscopic simulation programs, which model each vehicle as a separate entity, are usually expected to provide more accurate estimates of vehicle speeds and delays compared to analytical procedures, especially when the traffic impacts or roadway networks are complex. Therefore, the methodology for optimizing short-term work zone decisions based on simulation is introduced in this dissertation. With the same optimization algorithm 2PBSA, a simulation method, instead of the analytic method, is applied to evaluate the objective function. Two major barriers in applying simulation into work zone optimization include:

- Microscopic simulation is a quite time-consuming process for solution evaluation;
- Simulation results may not be precise when evaluating “bad” work zone decisions that may cause over-saturated conditions.

To overcome these limitations, two methods, namely a hybrid method and parallel computing, are proposed to increase solution search efficiency and reduce the computational burden imposed by the simulation process. The hybrid method, in which the analytic method is used in the first stage to identify promising regions for solutions while the simulation method is applied in the second stage, is tested through a hypothetical network. The result indicates the hybrid method can yield satisfactory solutions, which are close to simulation-based optimization results, but obtained with much less computation time. Experimentation also demonstrates the effectiveness of the parallel computing techniques.

(8) Summarizing findings based on sensitivity analysis

Based on the findings of sensitivity analysis aiming to examine the variation of key input parameters on the optimization results, general guidelines are developed to aid work zone management plan design:

- Proper lane closure tactics (e.g. scheduling work zone in time windows during which the remaining capacity is enough to accommodate passing traffic) can significantly reduce the work zone costs;
- Deployment of traffic impact management strategies, such as merge control and detour control system, can be beneficial and cost effective, especially on projects with high resource/labor idling cost, longer fixed work zone setup time, and tighter deadline.
- Detour control has great potential to mitigate the traffic impact and reduce project cost. Its effectiveness highly depends on road users' detour behavior model as

well as the physical and traffic characteristics of the mainline and alternative routes as well as.

- The paving strategy with the best durability or that with the lowest life cycle cost may not be the most cost-effective solution when the benefit-cost trade-off is considered. High traffic growth rate favors higher intensity strategy while high interest rate favors the opposite.

7.3 Recommendations for Future Studies

Despite the demonstrated capabilities of the proposed work zone evaluation and optimization models, we recognize that these models can be further improved. Possible extensions of the analysis and models developed in this study are desirable, as follows:

(1) Safety-Related Work Zone Impacts

In current studies, work zone impact on safety is measured by an estimated accident cost, which is calculated with a simple model relating accident rates to total user delays. In practice, the accident rates during road construction may be attributed to traffic volume, work zone length, duration of work, work intensity, lane closure strategy, lighting condition, truck involvement, use of traffic control devices, speed limit, and many other driver/vehicle/environment characteristics. The work zone decision evaluation model can provide better measure of safety-related impacts by incorporating an improved safety model based on best-to-know statistical analysis.

(2) Systematic Delay

We found that the systematic delay is a significant portion of total user delay especially when traffic volume exceeds capacity. Currently the systematic delay is estimated based on a regression model based on simulation results and the V/C ratio is the only independent variable in this model. It would be interesting to investigate what other factors contribute to the systematic delay and how.

(3) Simulation model

In the current study, simulation is used to estimate the user delays caused by work zone activities. In fact, many other Measures of Effectiveness (MOEs) can be obtained from the simulation outputs, such as density, speed, environmental effects and fuel consumption. We would seek to exploit more information provided by simulations in future research. In addition, finding ways to model different traffic control strategies, such as time-varying detour control system, within the simulation model can also expand the model's capability.

(4) Simulation-based optimization

In the hybrid approach proposed in this study, the analytic method and simulation method are used in different stages. Local search based on simulation is performed in a relatively good neighborhood obtained from the first stage. The algorithm might be more efficient when performing multiple analytic optimization steps between each simulation step instead of employing these two methods separately in two stages. Another possible way to improve the algorithm is to start the second-stage refined search from multiple local optimal solutions found in the first stage instead of

searching in only one good neighborhood. This procedure may require additional efforts spent on processing solutions saved in an elite archive.

(5) Multi-objective Optimization for Agencies and Contractors

The use of innovative time-related contracting methods (e.g. lane rental, and cost (A) and time (B) bidding) creates a new situation in which different decision makers may have different concerns so that a shared optimization model may not be suitable. To increase the applicability of the optimization tool by adopting different contracting methods or considering the objectives of different kinds of tool users, the proposed optimization models should be modified to reflect the needs of transportation agencies, contractors, or other involved stakeholders. For example, a multi-objective optimization model that simultaneously minimizes the total cost as well as the total project time may be more suitable for contractors to optimize their construction plan in Cost plus time (A+B) bidding provision. When lane rental is employed as a contract provision, it might be important to improve work zone cost model by adding working time-related cost and a "negative cost" representing financial incentives obtained from completing the project ahead of time.

(6) Uncertainty of Input Parameters

In current analytical work zone decision evacuation model, it is assumed that all parameters are accurate and deterministic. However, variation in parameters and collected data that may result in the uncertainty about the output is a well-known fact, especially in long-term analysis. The potential combined errors may impact the estimation accuracy and thus bias optimization results. Therefore, this issue should be

addressed in the future studies by quantifying the uncertainty of evaluation outputs and enhancing the robustness of the optimization method.

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